VOL. LIV. NOVEMBER, 1928. No. 9.

REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

LETTING CONSTRUCTION WORK BY COMPETITIVE BIDDING*

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Synopsis

This paper is an economic study of some of the successive steps taken in the preparation for, and the actual letting of, construction work by the method of competitive bidding. The parts or elements considered are:

A.—Preparation of the Contract, Specifications, Drawings, and Bond.
B.—Advertisement for Bidders.
C.—Information for Bidders, Questionnaires.
D.—Bidding Security.
E.—Awarding the Contract.

The endeavor is made to outline briefly a set of principles which, if followed, will assist in giving the owner the maximum value for the money expended in construction work and, at the same time, leave a profit for the contractor commensurate with the ability exercised in the performance of the contract and with the risk assumed in being bound to produce the work for the contract price. The owner supplies the funds, without which no construction work would be performed, and the contractor and his surety carry the inherent risks of the construction operations; it is uneconomical and against the best interests of all those engaged in the industry if any of these parties, or the engineer or architect, seeks to deny or withhold from any of the other parties the full value, protection, or profit that should rightfully accrue for his part in the undertaking. The owner, the contractor, the surety, and the engineer or architect are component parts more or less leaning for support on all the others. There is need of better harmony of effort. Perhaps this paper will lead to a better understanding among these parties.

Note.—Written discussion on this paper will be closed in **February, 1929.**
* Presented at the meeting of October 3, 1928.
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Introductory

In general, the paper supplements matter which has received the attention of many members in recent years.* Since 1917 the writer has reviewed a large number of contract documents, advertisements, bidding instructions, etc., on all kinds of construction work in all parts of the United States, and has been strongly impressed with the great diversity of methods in vogue on matters concerning which any group of engineers or architects should reach practically the same conclusions. Many of these procedures might affect the contract price to an amount of, say, 5 to 25%, or even more.

To illustrate: An engineer received bids on many millions of dollars worth of public work without even drafting a form of contract. The specifications and drawings were very sketchy, and the work was let to a company that tendered an informal bid on a new process which had never been tried on work of this size and character, and on which bids had not been requested. In reality, the contract was let without competition and all the formal bidders had their trouble for naught. The high character of the engineer precludes any collusion; but it was a queer way to expend the public's money.

In many cases the engineer and the owner word the contract so that the contractor becomes financier as well, because he must accept bonds or warrants at par in lieu of cash. Naturally, he must conclude a binding agreement with some strong financial party for the discount of the paper before he can prepare his bid. The owner may think he is borrowing the money at a nominal interest rate, whereas he may actually be paying a very much higher rate, but nobody but the contractor, his banker, and his surety knows it. Another variation of this is compelling the contractor to accept the unsold portion of securities issued to finance private construction. By this vicious practice many a contractor has been forced into bankruptcy as the market for the securities is saturated before he gets his securities and he cannot raise the funds needed to meet his expenses.

A certain architect prepares his contract, specifications, and drawings so that the bidders know exactly the amount and kind of work and materials required, hence the bids are always low and close. He gets the most for the owner's dollar. Another architect, doing the same class of public work, is known to furnish incomplete drawings and specifications and may change one without the other. They leave much to the imagination, hence bids vary greatly. Contractors who have done work under him are seldom the low bidders on new work. One such experienced contractor was the second bidder on a \$1 500 000 job and \$30 000 above the low bidder. The latter got the contract and collected more than \$100 000 "extras" at completion. At times, it is doubtful whether he expends the owner's money economically.

* The Society's Special Committees on Construction Contracts and on Engineering Contract Bonding have been engaged jointly with others in drafting the "Standard Contract for Engineering Construction," and the "Standard Questionnaires" used to determine the responsibility of a contractor, *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, pp. 223 and 259, respectively; also, some of the members assisted the "Interdepartmental Board of Contracts and Adjustments," U. S. Bureau of the Budget, in preparing the standard contract forms now used by the United States Government; Frank T. Sheets, M. Am. Soc. C. E., has presented a paper on "Qualifications of Contractors on Public Works," *Proceedings*, Am. Soc. C. E., April 1928, Papers and Discussions, p. 1021; and Ward P. Christie, Assoc. M. Am. Soc. C. E., a paper entitled "General Contract System *versus* Segregated Contracts," *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 395.

A.—Preparation of the Contract, Specifications, Drawings and Bond

Many contract forms give eloquent testimony to the fact that their authors never analyzed them. Inconsistencies abound largely because the same subject-matter is treated in several places. Many contracts are too verbose. Frequently, similar matter is given in the instructions to bidders, the contract, and the specifications. To express the same thought twice in the same language is useless repetition and to express it in different languages is to run the risk of confusion. Therefore, the following principle is evident:

(1) Treat any subject in only one place in the contract documents.

Nobody can see very far under water, or into the earth. As the owner may change his mind after the contract is signed, every contract should be drawn with the ideà that several changes will probably be made. Changes lead to claims for extras; hence the document should be worded so that any change is immediately recognized by each party, and an understanding regarding it reached. Even after the contract has been signed, the parties remain "free agents" to set up a supplemental contract and thus many claims for extras are pressed. If a contractor presents a claim months after the work has been performed, submitting a mass of records favorable to his position, the owner and his engineer are at a disadvantage because they, not knowing of the claim, have failed to gather evidence on the case.

In one contract form used on a large volume of public work there are no clear-cut distinctions between (*a*) increase in quantities to be paid for at bid prices; (*b*) "cost-plus" work; and (*c*) additional work at a price to be agreed between the engineer and contractor. Probably the unwillingness of contractors to start trouble with those who can give them subsequent work, plus the fair-minded interpretations of the engineers, explain why trouble has not caused this form to be clarified. Hence, the principle:

(2) Recognize that changes are inevitable; provide a means for promptly coming to an agreement regarding them; and require that an "extra" must be promptly declared.

Whenever doubt exists as to the exact volume, kind of work, and materials, the contractor is apt to estimate the most expensive method or amount. He cannot afford to do otherwise. Any uncertainty in the contract leads to either a higher bid than is economic or to a claim for extras, depending on whether the contractor is called on to produce at a cost less or more than he figured on. The phrase, "as directed by the engineer (or architect)", has cost owners a pretty penny in the past. A well thought out and detailed plan always indicates to the capable contractor that the engineer is competent and one with whom there is but little chance for disagreements. The next principle naturally follows:

(3) Every time an uncertainty is removed the price is lowered.

Many contractors are fully competent to supply an expert knowledge on certain construction matters far beyond the ordinary working knowledge of the engineer or architect. Often the results desired are specified instead of

the sizes, methods, and materials; but it is unfair to hold the contractor responsible for both. To do so is to make him and his surety guarantee that the engineer or architect knows his business. The best current practice tends to be specific. This gives the contractor a more exact job to estimate, also places all bidders on an even competitive basis. Therefore:

(4) If possible, specify workmanship and materials instead of results, but never both.

Reasonable tests or guaranties that the owner is getting the results desired are proper when results are specified, but unfair if workmanship and materials are delineated. If the owner decides to use a special process or product, let him accept the guaranty of those promoting it instead of compelling the contractor to assume the responsibility as is so often done. Maintenance guaranties on roads and pavements are manifestly unfair because, assuming the workmanship and materials are all that they should be, no one can estimate correctly the volume and kind of traffic the road must carry later. After the contractor has completed the work in accordance with the terms of the contract his responsibility should end. Guaranties that run for a term of years increase the price and decrease the number of bidders as contractors do not wish to carry an open liability. Accordingly:

(5) In general, do not require guaranties from contractors where workmanship and materials are specified. If required where results are specified, limit the guaranty to those things directly under the control of the contractor.

Why ask the contractor to assume a risk that might better be taken by the owner? The financing of contracts, the rental of a suitable operating space, the depth of foundations that may be required, and many other matters affect the bid price. The engineer may have several months to study the proposed construction, whereas the contractor has only a relatively short time to decide what effect, if any, these things will have on the cost. In many cases, it will pay the owner to remove some of these uncertainties in advance of receiving bids. Therefore:

(6) It is uneconomical to hire a contractor to carry a risk which the owner can carry at a less cost.

Most State lien laws compel the owner to meet unpaid labor and material bills if the contractor is unable or unwilling to do so. A sub-contractor's bill falls in this same class. These lien laws vary among the several States. Many are grossly unfair and against the best interests of the construction industry, and hence a joint conference has been drafting a uniform statute. Lien laws are not enforceable against a State or the Government, so it has been the custom to give Government contractors the same protection they would obtain in private work, by inserting a clause that they shall pay these bills and also by stating in the bond that the surety is liable. Labor really does not need protection because it will not work unless payrolls are promptly met, and whether it is desirable to guarantee the bills of those supplying materials is open to question. This certainly makes it easy for the irresponsible contractor to buy materials and place sub-contracts.

In all States except one or two, contractors have the full right to take money paid on one contract to retire indebtedness incurred on another, to play the stock market, or otherwise to expend it—and some of them do so. In the few States which word the contract and bond liability so that the surety is not liable for unpaid labor and material bills, the cost of a contract bond is only about two-thirds of what it is in the States where the liability is covered by the bond, because the unpaid bills generally represent a considerable part of the loss assumed by a surety when a default occurs. Hence:

(7) On State and Government contracts, removing the surety's liability for unpaid labor and material bills, will undoubtedly lead to a reduction in the premium charge for bonds.

Plant or equipment (the words are used interchangeably) is something needed by the contractor for construction but not left in the completed structure. Usually it does not wear out on one job and frequently its cost when starting a new kind of work will equal a considerable percentage of the entire contract price. There is no reason why a public works commission should be more interested in seeing that the seller of the plant receives his pay than the seller of a piano, automobile, or anything else purchased by the contractor. Still, a form used in many important public contracts provides that the contractor must pay all equipment bills and that otherwise the surety is liable. In fact, many forms go so far as to state that if the contractor does not pay these bills the owner will do so and charge the expenditure as a payment on the contract. Thus, nothing may prevent the irresponsible bidder from getting all his bills for labor, material, and plant guaranteed. He trusts to luck that he will make a profit, but cannot lose much because he is not worth much. The best way is:

(8) Remove all protection to sellers of equipment, as this gives irresponsible contractors an unfair credit.

In many contract forms the drawings cannot be listed or identified, and many drawings lack a date and title which ties them to a specific contract. An individual, a co-partnership, and a corporation each must sign a contract in a different way, but many forms do not recognize this fact nor attempt to record the status of the party signing. Sometimes the contract price does not appear in the contract, but a reference is made to a bid having been tendered. Frequently, no mention is made as to which alternate bid is accepted. This lax way of letting construction work is not limited to that handled by inexperienced engineers and architects as much of it comes from supervising officers that know, or should know, a better way. An engineer fails to discharge his full professional obligation, unless he sees that the contract form and drawings will cause his client, the owner, no embarrassment if a suit is brought. Hence:

(9) Prepare each contract with the idea that it may go into Court.

By a clause in most contracts, the contractor and his surety guarantee to save the owner harmless from all claims arising from the construction operations, patent infringements, etc. Many of these clauses are decidedly unfair because they try to make the contractor responsible for patent infringements

on methods or material specified by the owner as well as on those for which the contractor himself is responsible. This leads to the principle:

(10) Do not word the "save the owner harmless" and "patent" clauses so broadly that the contractor must assume risks beyond his control.

There is a decided advantage in binding together the contract, general conditions, specifications, reduced scale copies of the drawings, bond form, etc., as many persons are properly interested in every construction work of any size, and should have the privilege of receiving the full information. Material men and sub-contractors should be bound to the main contractor with all the obligations that bind him to the owner; the terms of payment may affect the price bid by a sub-contractor; and the surety is interested in learning the exact conditions of all parts of the contract, specifications, and bond before agreeing to assume the suretyship. The best way is to give all of them the contract documents bound together; when separated, there is always the risk of submitting only a part of the proposition.

It is difficult to hold a sub-contractor or material man when he has made a losing contract, and he is apt to claim the contract voided if the method of making payments deviates in the slightest from what he had in mind when the agreement was set up. Details regarding the payments generally appear in the general conditions and these should always be attached to the specifications when copies of the latter are given out. Contractors experience grave financial difficulties at times because of having to pay 100% to material men while receiving only a partial payment from the owner covering the same items, and all because the material men bid on the specifications only, instead of the complete contract documents. It is not sufficient to state in the specifications that a certain standard form of contract and general conditions will be used, as it is always possible to insert a typed page in such a form; also, all such forms have places wherein written entries will be made. Therefore:

(11) It is desirable to bind in one document the contract, general conditions, specifications, and bond form, also small-scale drawings.

Constructions supervised by engineers are in general partly paid for monthly by unit prices applied after measuring the actual work performed during the preceding month. Constructions supervised by architects are generally paid for on a lump-sum basis and, as no accurate unit quantities have been computed, the matter of monthly payments is not so easily determined. Often the contractor submits a monthly statement of expenditures for work and materials which when approved by the architect becomes the basis for payment. This method works well if the contractor is making a profit, but if a loss will occur it may lead to the over-payment of the contractor, and the owner may be holding as retained percentage his own money instead of funds already earned by the contractor. A much better practice is to have the contractor as soon as the contract is signed submit a schedule of all the component parts of the work with prices attached, the total of which will equal the lump-sum contract price. Thus:

(12) On all lump-sum contracts a quantity survey, with adequate prices attached is needed to assure the correct computation of the partial payments.

The "retained percentage" is a fund earned by the contractor but withheld by the owner as a safeguard in case of default. For many years the established practice has been to keep back a certain percentage, say, 10%, of each amount earned. During recent years some contract forms are worded so that the percentage is only withheld from payments on the first half of the work. In such cases, just before completion the owner would hold about 5% of the total contract price as against about 10% under the older method. As a matter of fact, creditors push a contractor harder near the end of a large job (perhaps realizing that a loss is imminent) than during the earlier stages of the work when the contractor may be optimistic regarding a profit.

A bank that extends a large credit early in the operations will insist that the indebtedness be reduced as the work progresses and will compel the contractor to return a considerable amount from the monthly payments, or to assign payments. The bank is in a position to make a good guess as to conditions near the end of a contract and will always turn this knowledge to its own advantage. Most contractors on large work must make a considerable initial investment when starting a job before the payments are sufficient to carry the work, so the owner runs less risk in making liberal payments at the start than near the end of a job. During the time an existing contract is nearing completion a contractor may be just starting a new job, and this is another reason why the owner needs to retain the full percentage near the end of the work. To sum up,

(13) More defaults occur near the end than near the beginning of a contract, therefore the owner needs to retain a greater reserve near the end than near the beginning of the work.

Any reputable surety company desires to be kept fully advised on the progress of the work covered by its contract bond. At times, it can exert a considerable influence on the contractor to keep him moving along to completion, and it will refuse to bond him on new work if the old work is going badly. It would serve the best interests of all concerned if the engineer or architect would promptly notify the home office of the surety of unsatisfactory progress or pending default. A good lawyer will always advise the owner to do so in the face of trouble, irrespective of any thing in the contract that indicates such a notice is not required. Especially is it desirable to obtain the surety's consent to any change in the method of payment or to any assignment of interest. Often it is difficult for the surety to obtain accurate information as to progress and this would be obviated if the engineer or architect would send to the Home Office of the surety a carbon copy of each monthly and final estimate. That is:

(14) Send to the home office of the surety a copy of each monthly estimate; also obtain from the surety its consent to any important change in the contract.

In most contract forms the authors fail to recognize that the "limits" of many kinds of insurance are just as important as the insurance itself. If a person using the streets is injured by an employee of a contractor very heavy damages might be recovered; also, the owner may be named as co-defendant with the contractor if a suit is started. Judgments for personal injuries are

becoming larger and larger each year and the owner may need the protection of a policy having "limits" of \$30 000 and \$60 000, whereas the "limits" of the policy actually obtained may only be the minimum \$5 000 and \$10 000. Therefore:

(15) Carefully determine the limits as well as the kinds of insurance protection needed by the owner and contractor.

An engineer may have several months in which to prepare a job for competitive bidding, whereas the bidders have a comparatively short time in which to become familiar with the terms and details of the proposed work. The bidder adds nothing for contingency on an obscure point if there is none. On unit price contracts it is desirable to take up each item or classification and tell all about it; thus, the entire job is analyzed for the contractor and in such a way that he can price each item with the minimum effort. With this method the customary clause should be included stating that the contractor is obligated to produce a complete structure; and that the costs of parts, general expenses, etc., not specifically enumerated in the unit items, will be borne by the contractor and are to be included in the prices bid on the regular contract items. The method will also prevent disputes on wrong classification during the construction period. Hence:

(16) On unit price contracts separately consider each bid item in sequence, telling where it is found, the amount at each place, with complete details.

If the drawings are not clear, the bidder cannot make a close estimate; also, there is a greater chance for disputes. In many cases more and better drawings will lead to lower bids. Isometric perspectives will often add considerably to the ease with which the entire project is assimilated by the bidder, and by the superintendent who later builds it. In other words,

(17) A little extra effort on the drawings is generally worth more than the cost.

The simplest form of bond is that guaranteeing the completion of the work in accordance with the terms of the contract, also saving the owner harmless against liens, claims, loss, or damage due to causes within the control of the contractor. This will give the owner all the protection he needs. The surety companies, however, seldom draft the bonds and frequently a form is presented that greatly increases the hazards of suretyship. Some forms double the premium charge without adding any benefit to the owner. The surety company always wants to know the exact wording of the bond before agreeing to assume the suretyship, and it is in the interests of conservative underwriting that the sureties should obtain this information. One standard form of contract states that the bond required will be "in such form as the owner may prescribe" and the surety is left in ignorance of what will later be presented. Through ignorance a "forfeiture" bond might be drafted where a simple bond form was intended.

A certain extensively used bond form states that the contractor shall "satisfy all claims and demands incurred for the same". If these words were followed by the phrase "for which the owner may be held liable", the liability

would be understandable; or if this extension of liability were worded instead "and pay all bills for labor and materials used on the work", a definite suretyship liability would be created. It is doubtful whether the person who drafted this form really had a clear idea of what he was doing. The bond is really paid for by the owner and it should be drawn so that he receives full protection and no one else. For many years a certain State has saved just one-half on its premium charges merely by adopting the proper bond form. The best practice, therefore, is to:

(18) Include the bond form with the papers given to the prospective bidder, and word it so that only the owner who pays for it will receive the protection bought.

The introductory part of many contract forms defines words like "guaranty", "surety bond", etc., and elsewhere in the contract are clauses describing the obligation of suretyship. The bond form itself generally describes this obligation. In reviewing a large number of contract documents the writer has noted, almost without exception, that the different descriptions of the obligation always disagree. From time immemorial bonds have been worded in a peculiar negative manner, and this language is not the kind that is readily inserted in a contract. Hence:

(19) The place to delineate the obligation of suretyship is in the bond and not in the contract.

While some financially strong contractors are able to obtain a fair volume of work without competition and at attractive prices (all contractors hope some day to be in this class), most work is contracted after competitive bidding has brought the price down. The plan works best when a surety bond is taken for the owner's protection. This is generally required when public money is spent, and the volume of bond protection on private work is constantly increasing.

When a surety issues a contract bond for a contractor it extends a credit to him because it increases his financial strength for the benefit of the owner in relation to the work bonded by an amount equal to the penal sum of the bond. A contract bond does not guarantee the owner that the contractor is responsible although many of the criticisms aimed at the sureties in recent years have assumed this. The obligation of suretyship is fully discharged by a monetary payment made to the owner after he has suffered a loss caused by the contractor failing to discharge his contractual obligations. It is necessary, however, for the surety to determine the contractor's responsibility for its own protection before assuming the contract bond obligation. Failing to do this, the surety will suffer the inevitable losses that go with careless underwriting. Others may form an opinion on the responsibility of a contractor, but they do not back the opinion for the benefit of the owner as does the surety company when it writes a contract bond.

Many times during recent years the surety companies have been told that they should render an inspection service along with the bond protection so that the owner will receive the surety's guarantee that the work is being performed strictly in accordance with the plans and specifications. This would

trespass on the professional services of the engineer or architect and is not a function of suretyship. It is an unworkable scheme.

Surety companies are constantly dealing with claim cases and their investigations generally disclose the causes of failures. Strange as it may seem, the inexperience of the contractor and the inadequacy of the contract price, while important elements, are not the principal causes of defaults, as the sureties pay many of their largest losses on contractors who are experienced and apparently were very well qualified when the bonds were written. Over-extension, or taking on more work than is warranted by the contractor's organization, plant, and financial strength, is probably the greatest single cause of failure. Many contractors plunge on real estate and otherwise tie up their capital in speculative investments where it will not work for them when needed to turn a financial corner. The defaults of important sub-contractors or failure to get labor and materials for the prices estimated, have caused many contractors to fail. It has even been stated by some contractors that the high cost of educating young engineers and architects in the technique of construction work has kept many from their estimated profits. There are many ways in which a contractor becomes irresponsible after the contract is signed, and, realizing this, the surety companies when re-letting defaulted contracts almost invariably require for their protection contract bonds executed by other sureties. They know the protection is worth more than the cost of the bond. It is a fact, that:

(20) The owner will generally receive the maximum value for his expenditure by letting work after fair competition and requiring a contract bond for his protection.

B.—Advertisement for Bidders

The purpose of the advertisement is to attract bidders. Therefore, it should be worded so as to show the prospective bidder whether or not he should spend the time and money necessary to investigate the project. A glance at the advertisements in any technical publication will prove that many of them fail to meet this requirement. A contractor will probably make a lower bid when the job fits his organization and plant because he knows just what the costs have been on similar work, and he may have just the plant needed. It is expensive to investigate prospective work, and this "over-head" is only paid for from the jobs that are obtained. Anything that will reduce the "over-head" benefits the industry.

Recently, an engineering paper carried two advertisements side by side for bidders on public work. One briefly gave the location, described the work, and the amount of each kind to be done; also it mentioned that the details could be obtained at the site (which was remote from the head office of the organization letting the work) as well as at two other locations convenient to bidders. The approximate bid price could easily be computed and the prospective bidder would know at once whether he should investigate. The other job was located at least two days' journey from the office where the details could be obtained after depositing $25; no plans were available at or near the site, and it is thought that most of the prospective bidders were remote from

where the plans could be obtained. Whether the job will amount to $100 000 or $1 000 000, is left to the imagination. While the advertisement does give the date on which the bids will be received, it does not tell the place, whether at the site or at the distant main office.

After spending some time and money investigating a prospective contract, the contractor is apt to put in a bid, anyway, even although the job is larger than he can finance; therefore, the advertisement should indicate the probable contract price. The surety companies also would greatly appreciate this information because nearly every experienced contractor approaches his surety to ascertain whether a performance bond could be obtained, before putting in a bid on a large contract; and often the surety is asked to pass on the matter before the contractor is in a position to make his own approximate estimate.

The surety companies subscribe to engineering publications largely for the advance information to be found in the advertisements to bidders, and telegraphic submissions of bond cases often refer the surety to certain publications for details. Every time a surety authorizes a bond that should not have been authorized or fails to authorize one that should be written, the best interests of all concerned suffer.

The following form of advertisement is suggested:

........(owner)........ will receive, publicly open and read, bids at its office at ....(address)........ at ....(hour)...., ...(date)......., for the construction of a ...., more fully described below.

(Here insert a paragraph giving location, accessibility (if desirable), and description in narrative or schedule form of the principal parts, items, or quantities. Include any pertinent facts regarding local conditions, the type of construction, the materials to be used, the size of buildings, etc., that will assist the contractor in determining whether his previous experience, plant, organization, and financial strength makes the work attractive to him.)

Approximate estimate of bid price between $............ and $.........., certified check or bid bond of $............, required, and performance bond will be $........... (or ....% of contract price). After .....(date)...... the plans, contract, and bidding forms can be obtained from above mentioned office, or from..

(Dated)...................... (Signed)..........................

The following principle holds:

(21) It is to the owner's advantage to word the advertisement so the bidder can tell whether the proposed contract will fit his ability, experience, plant, organization, and financial strength.

C.—Information for Bidders, Questionnaires

In the "Information for Bidders" is the place to state that the right is reserved to reject any or all bids and to tell the bidder where and how the bids will be received as well as what evidence, if any, he must present to qualify for the contract, should it be awarded to him. The contract later to be set up is supposed to be a fair "meeting of the minds"; hence, anything told the bidder in the "Information" that influences his conception of the proposed work, has much the same virtue as if included in the contract. It is believed the Courts have generally held this view.

Certain clauses in the general conditions of the contract, as on the bidding security, time allowed for signing the contract, etc., could with propriety be transferred to the "Information for Bidders", thus properly confining the contract proper to matter describing a method, detail, part, or condition of the consideration that one party gives the other. Hence:

(22) Many contract forms would be improved if clauses on subjects which are active before the contract is signed were placed in the "Information for Bidders" instead of in the contract.

Every prospective bidder should have equal privileges in obtaining information about local conditions and whether or not alternate or informal bids will be entertained by the owner. It is decidedly unfair to let formal bidders expend their time and money preparing bids and then to award the contract to some informal bidder. If the owner is dissatisfied with the method he has specified and wishes to consider another one, the fair thing is to reject all bids or postpone the letting until all bidders can submit amended tenders for the work. That is:

(23) Each prospective bidder should receive all the information given to any other bidder; and have the same opportunity to submit alternate or informal bids.

One of the best ways to minimize over-extension would be to give the prospective bidder a chance to limit the volume of new work he could receive at any letting, provided bids are taken on more than one job. The owner thus secures more competition per job and loses no economic advantage. The contractor is benefited because he has a better chance of getting a job when bidding on a large volume of work than on only the additional quantity that will fit his organization and ability to finance. Therefore:

(24) Permit bidders to limit the amount of new work that may be awarded to them at any letting.

Most constructions supervised by architects are built amid urban surroundings where materials like sand and crushed stone are staple commodities with fixed prices. Many jobs supervised by engineers are at remote places where even the water supply may influence the cost. All specifications call for "clean and sharp" sand; but many localities cannot produce such a material although the local sand has been used satisfactorily. The engineer will secure much lower bids if all local sources of materials are investigated in advance. The writer recalls a case where it is thought an advance expenditure of $500 by the engineers would have saved a very large sum, possibly several hundred thousand dollars. Principle (3) is also pertinent to this subject. The best plan, then, is to:

(25) Investigate local facilities and materials and furnish the information to all bidders.

Among contractors, engineers, and architects, there is a tacit understanding that the owner will proceed with the award within a reasonable time after receiving bids; but every now and then a contractor finds himself the low bidder with no definite idea of when, if ever, the contract will be signed. Such a delay is unfair as he hesitates to bid on other work; also his bidding

security is still in the hands of the owner, and it may be difficult or impossible to obtain additional credit with which to bid on other contracts. As long as the bid is outstanding it is a potential contract and it affects his "responsibility" for other work. It is only fair, therefore, to:

(26) Specify that within thirty days after receiving bids, the award will be made or all bids will be rejected.

In a few States the statutes require that the engineer's estimate of quantities and bid prices on highway work be made public in advance of the receipt of bids. This is against the best interests of the States, as it makes it too easy for inexperienced contractors to prepare bids. What may be a fair price for an experienced contractor with full equipment and a good organization may be wholly inadequate for a beginner. If the bidder is qualified to construct the work, he is qualified to prepare his own estimate. It is proper and desirable to tell the prospective bidder the approximate bid price within limits, but not to give him a detailed estimate or to ask him to bid on a percentage of the engineer's estimate as is often done. Therefore:

(27) Do not give prospective bidders detailed estimates of proper bid prices on road or similar work.

For building work, where lump-sum bids are generally required, bidders must estimate the quantity of each item before they can fix the price. This is a laborious operation on a large building, and the cost is added to the bid price. Why should the owner pay the expense of ten, or more, quantity surveys on one job when he can provide one survey to serve all bidders, and at the same time bring all bidders into true competition on the basis of price only? In many localities groups of contractors find many advantages in having joint quantity surveys made. Any such plan requires distinct drawings and specifications, as uncertainties are bound to be uncovered by those preparing the quantity surveys. Furthermore, corrections made before the work is offered for bidding, lead to lower bids. As a general principle:

(28) It is economic for owners to offer quantity surveys to bidders on building work.

Many advertisements for bidders are worded as if the owner were conferring a great favor on the contractor to permit him to bid—in fact, some owners charge the contractor a good fee for the plans. In reality, the favor is extended by the bidders because it costs more to prepare the bids than to provide contract documents. After a bidder has qualified it is to the owner's advantage to loan him without charge all the contract documents he really needs in order to prepare an estimate carefully. Making it easy and inexpensive to bid is bound to be reflected favorably in the price and a $100 expenditure by the owner for additional sets may return him thousands of dollars when the bids come in. Many parties as sub-contractors, material men, and sureties have a proper interest in reviewing the contract documents carefully before the bid is tendered.

Frequently, when deposits are taken, the time within which the plans must be returned is given as seven days after bids are received, and some advertisements state that all the documents must be returned with the bid. This is

unfair, especially to parties who may have given sub-bids. In one metropolitan district sub-contractors on a certain class of work had so many disputes with contractors that they announced a decision to retain the drawings and specifications until such time as their bids had been definitely rejected. They had been asked too many times to perform at the old prices after changes had been made by the owners subsequent to the receipt of bids. Until all bids are rejected or some bid is accepted, the matter is open; therefore, the return of plans should be based on a period starting from the award of the contract or the rejection of all bids instead of from the date bids are received. A 14-day period is suggested as a fair condition. To sum up:

(29) Be liberal rather than close as to the number of sets of the contract documents given to prospective bidders. Do not charge for these, nor ask for their return in less than fourteen days after the award or rejection of all bids.

Some contractors, apparently, get a great deal of satisfaction in bidding high without any desire to receive the award; others put in high bids merely because the engineer or architect asks them to estimate the jobs and they desire to remain in his good favor. Still others, having employed estimators, think they must keep them busy. A complimentary bid is disturbing and serves no real purpose. Often it causes difficulty for the low bidder to obtain a bond, or credit from his bank, because a few very high complimentary bids from contractors who are known to have a good knowledge of costs, make it appear as if the low bidder had greatly under-estimated the job and, therefore, will have a losing contract. Hence:

(30) Engineers and architects should not encourage contractors to submit complimentary bids.

Available local facilities have a considerable influence on the prices which must be paid for certain constructions. Engineers have been known to increase the prices on their own contracts by letting so much work within a short time that there is no real competition as each contractor having the requisite plant is reasonably sure of obtaining all the work he desires. A certain owner always tried to spend the large sum available at a rate about 50% faster than the contractors could work, due to limited facilities for labor, materials, and transportation. As a result bid prices constantly rose although the contractors failed to make profits on most of the work. A large number defaulted and others withdrew after suffering large losses. In other words:

(31) To obtain economic bids, do not let more work than can be readily performed.

The allowance of the customary week or ten days after the award within which the contract must be signed and the bond given is often much too short and fourteen days should be allowed instead. The surety companies are hard pressed at times to underwrite bond cases, secure re-insurance from other companies, etc., within the short time available. With the extra days a case could be presented to the surety by mail with all the supporting documentary evidence, giving the underwriter a clear-cut picture, instead of rushing the case through by telegraphic presentation. It is on the side of

conservative contract bond underwriting and does not really delay the actual beginning of the work because the contractor starts on his preliminary arrangements just as soon as he receives the award. Therefore:

(32) Allow fourteen days after the award for signing the contract and giving bond.

A commendable practice is growing; the prospective bidder is required to fill in a questionnaire form demonstrating his responsibility before he receives the bidding forms or is awarded the contract. It is unfair to encourage him to bid and perhaps later deny him the award, if low; also it is thought that a contractor who has too much work on hand to warrant his being allowed to bid will make less disturbance if disqualified before bidding than after finding himself the low bidder. The forms are still quite new and undoubtedly changes will be introduced from time to time. Few of them direct an inquiry regarding outstanding bids or the amount and status of other contracts on hand, and this is a serious omission because the volume of work is an element which enters into any determination of responsibility.

One form asks the bidder to describe the methods or plant he will use if awarded the contract. This is an unfair question to ask of an experienced contractor. If the engineer wants a certain kind of plant used he can specify it, but if the contractor has to produce a specified volume and quality of construction work he should be allowed the greatest latitude in the selection of methods or plant. The construction industry owes much to the pioneers who have dared to use new processes and thus have found a cheaper and perhaps a better way of producing an old form of work. Why should the contractor disclose his methods before the contract has been awarded to him, especially as all bids may be thrown out and his competitors may consider the new method at the re-letting? In short:

(33) Use questionnaire forms to qualify prospective bidders instead of to justify awards.

D.—Bidding Security

Bidding security is not generally required on private construction work; but on public work usually a deposit with the bid of either cash, Government bonds, a certified check or cashier's check, or a bid bond is required. At times, the bid must be accompanied by a letter or certificate issued by a corporate surety company stating that it will write the contract bond for the bidder should he be awarded the contract. The bidding security is supposed to be a guarantee deposited with the owner binding the bidder to execute the contract and file the required contract bond should the contract be awarded to him.

References to the bidding security are generally included in the advertisement and "Information for Bidders". In many proposal forms the clause stating the conditions under which the bidding security is deposited as a guaranty fund for the owner's protection is decidedly unfair because it provides that, if the contractor fails to execute the contract and file the bond after an award, the fund is forfeited to the owner. The measure of damage to the owner is the difference between the bid that was not executed and

that of the next, or some other bidder, who qualified. The owner after thinking so well of the contractor as to encourage him to prepare and tender a bid is treating him rather harshly to turn any possible misfortune to his own advantage.

The amount of the bidding security is generally about 10 or 15% of the contract price. It is desirable that the owner state a definite amount rather than a certain percentage of the bid price, because the bidder may not know what he will bid until just before the bid is tendered. The practice of some public work bodies seems to encourage the financially weak contractor to bid on work beyond his capacity as they graduate the size of the security required up to a maximum of, say $25 000, which sum would apply to any contract of more than a certain amount. In one case the maximum security of $15 000 was supposed to give the owner protection on a contract of nearly $500 000. A $50 000 bidding security would have been much more appropriate. It is best, therefore, to:

(34) Designate a lump sum for the bidding security in an amount from 10 to 15% of the probable contract price, and condition its receipt so that the measure of damage to the owner, if the bidder fails to execute the contract and file a bond, is the difference between the amount bid and the amount for which the owner may be able to award the contract within a reasonable time.

Many engineers, contractors, and surety companies believe it is desirable to abolish the use of bid bonds and hold strictly to the use of cash, Government bonds, or certified checks. The requirement that checks shall be filed is increasing and bid bonds may soon be a thing of the past. Almost all bidding checks are borrowed from a bank because they may be on deposit for some time after the bids are opened, and contractors seldom have enough surplus cash to permit a considerable sum to be taken from the business. This borrowing is really desirable as it brings them together at a time when the banker may give the contractor some good advice against taking on more work than can be easily financed. Many contractors after such talks have wisely decided not to bid. In all cases, then:

(35) Do not permit bid bonds as bidding security.

The letter issued by a surety company stating that it will write the contract bond if the bidder is awarded the contract is called a "bid letter" by the surety, and is handled by it just the same as a bid bond. All the reasons that make bid bonds objectionable apply equally to bid letters. The owner fixes the amount of the certified check and should make it large enough to supply the protection desired without trying to bring in a surety company to bear part of the load. It is in line with conservative underwriting if the surety gives no letter or certificate to the owner when bids are received, because seldom has the surety had time to give the case the full investigation needed. Therefore:

(36) Do not ask for bid letters.

E.—Awarding the Contract

Often bids have been prepared at considerable expense and it is not treating the bidders fairly unless they are accorded the privilege of being present

when the bids are publicly opened and read, as this is the only guaranty that there is real competition. Otherwise, the bidder is apt to suspect that the job is "set" for some favored contractor. Public opening is almost universal when public money is to be expended, but on private work there is still a considerable proportion of secret openings. It is claimed that often after a secret opening the better class of contractors are asked to cut their bids to meet the figures of irresponsible contractors who have nothing to lose and who therefore bid much lower than good construction work can be produced, and who may have been asked to bid for this reason. So flagrant has this practice become in certain localities that the reputable contractors have found it to their advantage to file duplicate bids, one with the owner and the other with some designated person who later gives each bidder a tabulation of the bids. Naturally, the owner does not get very far in "peddling bids" when the best contractors know all the details. So it is good policy that:

(37) Bids on public or private work should be publicly opened and read.

Scarcely any reputable contractors are averse to bidding on important contracts located in other places. It is generally considered strictly proper for contractors to bid on distant work and for owners to invite such bids. Frequently, however, an out-of-town bidder is low and a local contractor is next; then the greatest pressure is brought to bear on the owner to let the work to the local bidder under the plea that "all the money should be kept at home". Such an award would be unfair to the outside bidder who was encouraged to bid and who believed, tacitly perhaps, that if he was low, he would receive the award. If only local bidders are desired, the advertisement should state this condition. Even should the out-of-town contractor do the work, most of the money will stay at home because it is largely the local materials that will be used, also local labor; and the contractor only takes away with him the profit, if any, at completion. If he loses because he bid under the local bidder, the out-of-town contractor might, in fact, make a substantial contribution to the community. The rule should be:

(38) Award the contract to the lowest bidder if he is qualified, irrespective of what a local bidder has bid.

No owner wants his work performed by an irresponsible contractor, nor do engineers desire him to undertake contracts supervised by them. It is supposed to be the professional duty of an engineer to advise the owner regarding the low bidder and to urge the award to a responsible contractor.

Any contractor who is dishonest, tricky, and bears a bad reputation with those for whom he has done work is, of course, irresponsible no matter how many ill-gotten gains he possesses. Fortunately, there are not many such in the construction industry, and the contractors who are doing the greater part of the large yearly construction programs are men with just as high ideals and character as will be found in any other large industry.

However, merely being honest does not make a contractor "responsible" to undertake a large contract. In addition he should have ability as evidenced by his past experience and record; an organization suitable for the

proposed work; and sufficient working capital or net quick assets to finance the operations and reasonable additional costs. These qualifications should give a fair assurance to the owner (or a surety company) that the contractor is well able to complete satisfactorily (*a*) all the unfinished contracts on hand; (*b*) the new contract in question; and (*c*) all the contracts subsequently taken until the new contract has been completed and all bills paid. When a contractor gets into trouble on one job it soon spreads to all his contracts; therefore, the entire volume of work he is obligated to perform is a very important element in determining his responsibility although many do not give this element due weight. Assuming the contractor is otherwise qualified, his responsibility is determined by the ratio his financial strength bears to the total volume of work he must perform.

When the contractor is working well within the proper ratio, he should be a good risk for himself, as he will probably have sufficient assets to meet an ordinary financial strain if necessary, and so will continue in business. When he is over-extended or suffers bad luck, he may be forced into bankruptcy because credit quickly becomes afraid and runs away in the face of trouble—that is, all credit except the bonding credit which has to stay because a contract bond is a non-cancellable obligation. While the ratio of the financial worth to the total work should always be determined, so that it can be duly considered when judging the responsibility of a contractor, it is doubtful if any fixed ratio rule could be applied generally, because contractors vary in their ability to overcome difficulties and because many kinds of work are inherently much more hazardous than others. Probably no two persons will reach the same conclusion on responsibility although experienced surety company underwriters will frequently agree on how much work of a certain class a contractor should undertake at any one time.

Many surety companies have a basic ratio rule about as follows: Assuming the contractor is capable, experienced, and has all the plant and organization needed, he should have unquestioned net quick assets of not less than 10% and other assets of an additional 10% of the total volume of work to be performed. Unusual ability and a kind of work that is not hazardous might lead the surety to reduce this ratio, while work of a hazardous kind, or under unfavorable local conditions, would lead the surety to desire more working capital in proportion to the volume of work.

Many engineers, architects, and contractors do not recognize any ratio rule in determining responsibility, but classify a contractor according to the size of the contracts previously performed by him without any consideration of the number he is qualified to handle and finance at any one time. A contractor who has a large plant and little working capital is a poor risk, even on a contract that is well inside the ratio, and is taken when he has no other work on hand, because, in all probability, he will soon take on other contracts in order to keep all his plant at work and then will have too much under way at one time. Too much plant without corresponding net quick assets has wrecked many contractors, especially in the road-building business. A contractor can often make more money closely supervising one contract than in trying to

handle several, some perhaps at a distance, and this has been proved over and over again in the financial statements filed with surety companies.

While it is comparatively easy for an engineer to follow the progress being made on a contractor's work on hand, it is not so easy to analyze his financial statement and determine what allowances should be considered in arriving at the net quick assets and net worth. All parties habitually dealing with credits will disallow or greatly scale down all obscure items; contractors, unfortunately, have no standard method of keeping their accounts or preparing financial statements, so that many asset items cannot be evaluated without the supporting information.

Anticipated profits are frequently included and many contractors think every dollar expended on a contract is bound to come back accompanied by the estimated profit. Notes receivable may have been given in payment for a junior interest in the co-partnership or corporation which conducts the business and these really add nothing that has a quick sale value. Stocks and bonds may appear at inflated values. The contractor or a certified public accountant can say under oath that the statement shows the true condition of the books, but such an oath does not necessarily determine what net quick assets or net worth should be allowed. All these considerations lead to the principle:

> (39) Ascertain the ratio between the contractor's financial strength and the volume of work he is obligated to perform. Use this as an aid in determining his responsibility before awarding the contract.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PUMPED-STORAGE HYDRO-ELECTRIC PLANTS

By William W. K. Freeman,* Assoc. M. Am. Soc. C. E.

Synopsis

The Rocky River hydro-electric development, recently completed near New Milford, Conn., is the first modern plant in America to pump water into a reservoir with off-peak power, in order to produce on-peak power later. In connection with this project, a search of engineering literature was made for data on similar plants built elsewhere. The number of such plants was discovered to be forty-two. Ten other plants have been proposed in Europe (including one 600 000-kw. installation) and thirty-six in America. The rapid increase in size of units and total installations and the number of plants built in Europe in the past few years, when considered with the building of the first large plant in the United States, make it appear that America is on the threshold of a new phase in hydro-electric development.

The principle of operation of such plants is therefore explained, with a discussion of their economic justification; a summary is given to point out the extremes in physical features and the advanced stage of development of this type of hydro-electric development; and brief descriptions of the most interesting pumped-storage plants are included, particularly of those which are adaptations of existing developments. Their fundamental cycle of operation is discussed in the Appendix.

I.—General Description

Growth and Size.—An increasing number of American hydraulic engineers are becoming interested in the type of hydro-electric development in which water is pumped into a reservoir with off-peak energy in order to produce on-peak energy later. Few of them, however, know that this has been done for nearly half a century; that forty-two plants have been built to date

Note.—Written discussion on this paper will be closed in February, 1929.

* With New England Power Constr. Co., Boston, Mass.

(Fig. 1); that twenty-one of them have been built within the past five years; and that their evolution has progressed to the point where 34 000-h.p. pumps are being installed and a total pumping capacity of 250 000 kw. has been seriously proposed (see Tables 1 and 2). Ten more plants have been proposed in Europe, where practically all this development has taken place. In the United States, however, the first modern plant of this type has recently been completed by the Connecticut Light and Power Company on Rocky River, near Milford, Conn., although thirty-six more have been proposed, among them being one to pump 5 000 sec-ft. against a 90-ft. head over a divide.

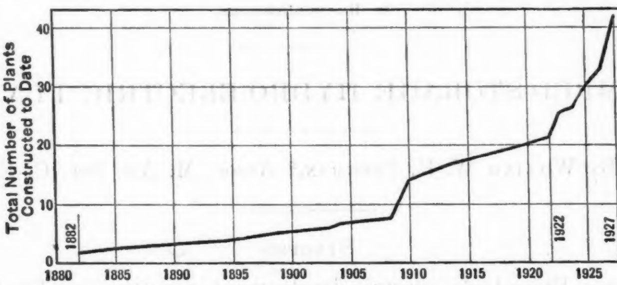

FIG. 1.—HYDRO-ELECTRIC PLANTS WITH STORAGE RESERVOIRS FILLED BY PUMPING.

Those who are in close touch with the situation regard the Rocky River development as the first of a large number of such plants in this country. The inevitable human inertia due to a conservative attitude toward the transplanting of a European idea to American soil has now been overcome and with the building of the first large plant in this country the development should be rapid. It is timely therefore to explain the principles of operation and to show the wide range of conditions for which these plants have been built, as well as to stress their advanced stage of evolution.

"Hydraulic accumulator" is a name that has been given such plants, ill-advisedly, in the writer's opinion, because an hydraulic accumulator is a system used principally in rolling mills for supplying jacks with water under high pressure. A preferable term is, therefore, the more graphic "pumped-storage hydro-electric plant".

PRINCIPLE OF OPERATION

The principle on which these plants operate is that of building a storage reservoir where the natural conditions are favorable for the economic storage and utilization of water even if the natural inflow into the reservoir is negligible, provided there exists, a short distance away and at a lower level, a supply from which water can be pumped into the reservoir. This pumping is done with low-cost off-peak energy generated at steam plants or run-of-stream plants, in order later to obtain high-value peak energy from the storage reservoir. The process of using power to pump water into a reservoir and drawing it out again to obtain power would be uneconomical except for the fact that electric energy must be produced at the time the consumer requires

it. The marginal cost of off-peak energy from steam turbo-generators which would be idle or partly loaded if it were not for the pumping load of a pumped-storage plant, is quite low; and the marginal cost of off-peak energy from water which would otherwise be wasted over the dams of run-of-stream plants (called "surplus hydro-electric energy") is practically zero. These conditions tend to make it economical to use this equipment, which would otherwise be wholly or partly idle, but on which nevertheless the fixed charges would be continuing, to pump water into a reservoir to produce power later when required, the arrangement resembling an electric storage battery in its operation.

Types of Plants

Storage Type.—There are three types of plants, the storage development and two variations of it—the pondage and the part-head types. Rocky River is a plant of the storage type (see Fig. 2), in which two principles of operation are used. By the first, water is drawn from the reservoir to generate on-peak power in this plant only, and is replaced by an equal quantity within the same day or week, pumped into the reservoir with off-peak power. In this case, there is no net depletion of the reservoir, and the plant is said to operate on the pondage (that is, short-time storage) principle.

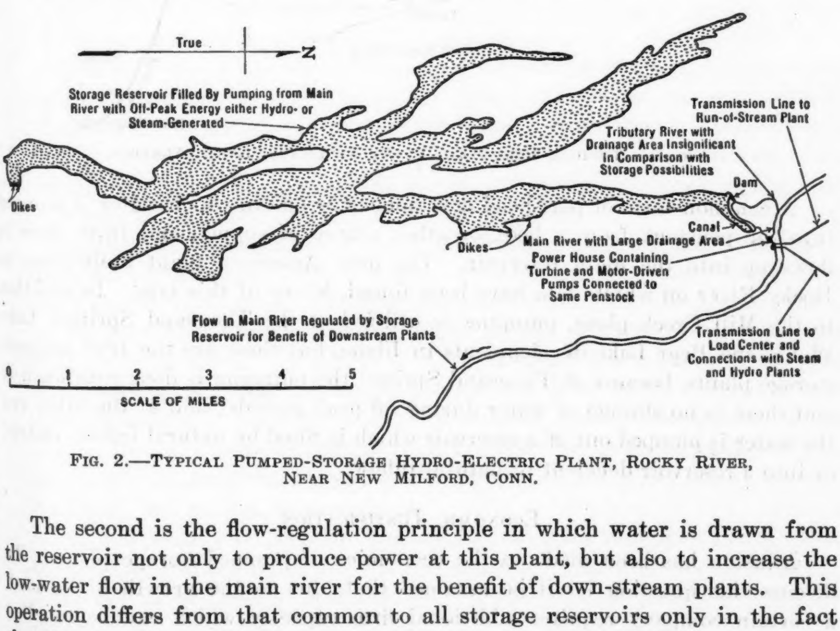

FIG. 2.—TYPICAL PUMPED-STORAGE HYDRO-ELECTRIC PLANT, ROCKY RIVER, NEAR NEW MILFORD, CONN.

The second is the flow-regulation principle in which water is drawn from the reservoir not only to produce power at this plant, but also to increase the low-water flow in the main river for the benefit of down-stream plants. This operation differs from that common to all storage reservoirs only in the fact that the reservoir is filled by pumping because the natural inflow is too small.

Pondage Type.—Nearly four times as many pondage plants as storage plants have been built. In this type only short-time storage is economical and the feature of releasing water to regulate the flow of the river below the reservoir, as described for the storage type, is entirely absent. Some have

pumps of enormous size; as, for instance, the Hengstey plant now under construction, which has 34 000-h.p. pumps. Among the developments of medium size are some reservoirs formed by dikes on hillsides, while a few of the reservoirs are so small that they are made of reinforced concrete.

A number of the plants of this type have two reservoirs, one forming the forebay of the plant and the other the tail-race. In this case the only additional water used, once the lower reservoir is filled, is that required to replace leakage and evaporation. This type of plant extends the possibilities of hydro-electric development to locations where it has never been considered, because practically no natural flow of water is necessary. It requires only suitable natural or artificial pools at different elevations, with the proper hydraulic works.

Part-Head Type.—As illustrated in Fig. 3, the third or part-head type pumps water against a certain head to make it available through a higher head.

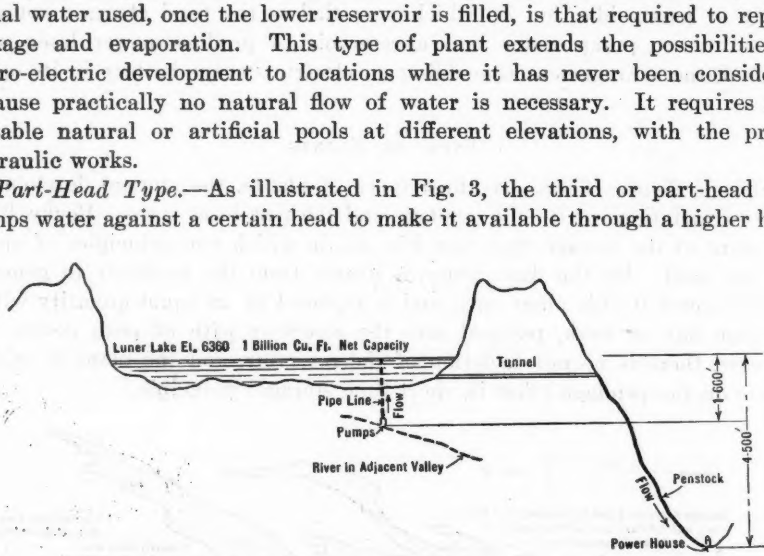

FIG. 3.—TYPICAL PART-HEAD PLANT AT LUNER LAKE, AUSTRIA.

A common form of part-head development is one in which water is pumped into the penstock from a lake, or other source of supply, and from there it flows up into the main reservoir. The only American plant built prior to Rocky River on which data have been found, is one of this type. In addition to the Mill Creek plant, pumping is utilized at the Thousand Springs, Lake Waha, and Bear Lake developments in Idaho, but these are not true pumped-storage plants, because at Thousand Springs the pumping is done continuously and there is no storage of water during off-peak periods; and at the other two the water is pumped out of a reservoir which is filled by natural inflow, instead of into a reservoir deficient in natural inflow.

ECONOMIC JUSTIFICATION

Allusion has been made to the economics of pumped-storage plants. To discuss this question let it be assumed that, due to the growth of its load, a certain company requires additional firm capacity which can be supplied by the enlargement of its steam plant or by the addition of a pumped-storage plant to its hydro-electric system, in which there are some run-of-stream plants. The net value of the hydro-electric plant, therefore, is determined in the following manner. Hydraulic computations for a certain installed capacity show:

(1) The increase in the firm capacity of the system due to the building of the pumped-storage plant;
(2) The increase in the primary energy;
(3) The decrease in the secondary energy produced by the hydro-electric system due to storing water during periods of higher flow; and
(4) The secondary energy used for pumping.

It is considered properly conservative to assume that all pumping both for pondage and for storage will be done by steam-generated off-peak energy (although a part of it may be done by water otherwise wasted over the dams of run-of-stream plants). It is likewise conservative to omit the saving due to the plant's production of secondary energy except from the natural inflow into the reservoir, as discussed in the Appendix.

The value of the hydro-electric plant is the sum of its "capacity value" plus its "energy value", and is measured by the fixed charges and the production costs, respectively, at the alternative steam plant. The meaning of the term, "capacity value", is best explained by an illustration. Thus, if a pumped-storage plant adds 75 000 kw. of firm capacity to the system, and the alternative addition to the existing steam plant would consist of two 40 000-kw. units at a cost of $9 600 000, the fixed charges at 11% would be $1 060 000, or $13 per kw. per year. The capacity value of the 75 000-kw. hydro-electric plant at $13 per kw. per year is, therefore, $975 000 per year.

Various methods are used to calculate the energy value of a hydro-electric plant. By one method the value of the energy is taken as the production cost of the same energy at the alternative steam plant. For a pumped-storage plant the energy value is found by subtracting from the value of the primary energy, the sum of the values of the decrease in secondary energy due to storing water plus the secondary energy used for pumping.

Against the capacity value plus the energy value of the hydro-electric plant, are set the fixed charges on the investment in the pumped-storage plant plus its production costs. The difference represents the annual saving through the building of the pumped-storage plant instead of an addition to the steam plant, or *vice versa.* In addition to these items there are also frequently others, such as, fixed charges on the unused capacities of the alternative installations before they are fully loaded; investment and operating costs of transmission lines; relative reliability; and the addition of an independent source of supply. Some of these may easily be evaluated; while others cannot be evaluated, but should be considered.

As may readily be seen for any specific case, the economic justification of the pumped-storage plant lies in its capacity value even more than in the difference in production cost between off-peak and on-peak steam-generated energy.

General Economic Considerations

The writer hesitates to try to formulate general statements as to what conditions make plants of the three different types economical because of the infinite variations there can be in topography, market, competing power, and

surplus power for pumping, all of which influence vitally the economy of any project.

In general, however, it may be said that the conditions that make ordinary hydro-electric plants feasible make pumped-storage plants feasible; for example, low investment cost per kilowatt of firm capacity; high cost of competing fuels; large and concentrated system loads, etc. Although the pumped-storage plant labors under the disadvantages of the added cost of the pumping equipment and operation, it has certain compensating advantages over ordinary storage plants, some of which may be enumerated as follows:

1.—It is comparatively independent of rainfall and drainage area—although, of course, a high average annual rainfall on a large drainage area means less pumping.

2.—In more settled areas the land and other damages are apt to cost less for a given reservoir capacity because the plant is on a tributary instead of on the main river.

3.—Since, for a given reservoir capacity, the drainage area is smaller, the provision for floods is less expensive, if not absent.

4.—The plant that pumps from a large river, or from a pool, is valuable as a reserve plant because water released in an emergency can be replaced by pumping.

5.—Finally, among the most important advantages is the low cost of adding pumping and power equipment to an existing development to obtain more firm or reserve power. Three examples of such plants are given in Part 2 and it is the writer's opinion that considerable activity will be shown in the United States in the adaptation of existing plants to the pumped-storage principle; the adaptation of one plant has already been seriously proposed.

How far these favorable features outweigh the added investment and operating cost depends on local conditions. Since experience with this type of plant is necessary to find the limit to its field, it is hoped that if there is any discussion of the economics of pumped-storage, those who have data on uneconomical projects investigated, will present them.

SUMMARY OF DATA

There are 6 storage plants, 21 pondage plants, 7 part-head plants, and 8 plants not classified as to type due to the scarcity of data on them. The following summary of the outstanding features of the plants built to date or under construction (see Table 1) is based on incomplete data and is, therefore, necessarily approximate.

Salient feature.	Location.
Earliest, 1882 (hydro-mechanical until 1891)	Zurich, Switzerland.
Largest reservoir, net capacity, 6 000 000 000 cu. ft. at present; 7 000 000 000 cu. ft. ultimately	Rocky River, Connecticut.
Smallest reservoir, capacity, 200 000 cu. ft.	Tübingen, Brunnenmühle, and Zweribach, Germany.
Highest head in full-head development, 2 900 ft.	Tremorgio, Switzerland.
Lowest head, 80 ft.	Cornabbia, Italy.

Salient feature.	Location
Highest reservoir elevation, 7 710 ft.	Rhône Valley, Switzerland.
Lowest reservoir elevation, 430 ft.	Rocky River, Connecticut.
Most powerful pump, 34 000 h.p.	Hengstey, Germany.
Largest pump in capacity, 480 sec-ft.	Hemfurth, Germany.
Highest pump efficiency, 87%	Rocky River, Connecticut.
Most powerful turbine, 45 000 h.p.	Hengstey, Germany.
Greatest installed turbine capacity, 180 000 h.p.	Hengstey, Germany.

Proposed Plants

The biggest project on which data have been published is that for which a grant has been applied by the Rhine-Westphalia Electric System and the General Electric Company (see Table 2). They proposed ultimately to build a 600 000-kw. plant on the Our River, in Germany, with a main dam 350 ft. high, which alone will have 920 000 cu. yd. of concrete. With this large volume of concrete is to be compared the 660 000 cu. yd. at Conowingo, Md., a run-of-stream plant with an ultimate installation of 396 000 kw. At the Our River development there are three rivers, on one of which there is a reservoir with a capacity of 300 000 000 cu. ft. for the Stolzenberg plant which operates under a 330-ft. head. Into this reservoir the Sauer River will be diverted and here will be installed, in the power-house, pumps with an ultimate capacity of 250 000 kw. to fill the main reservoir of 28 000 000 000 cu. ft. on the Our River. The plant will replace the 25% reserve capacity of the companies' existing steam plants which will be used 6 000 instead of 3 000 hours per year, and it will produce 500 000 000 kw-hr. per year, of which one-third will be from the natural run-off and the remainder from pumped-storage. Most of the energy for pumping will be produced at steam stations.

A proposal of the magnitude of the Our River project indicates what, in time, may be expected in this country. An old report on Rocky River considered a 2 100-kw. installation to use the natural flow of the river, while the ultimate proposed now is twenty-three times as great, due to the utilization of the large reservoir for pumped-storage. It has been said of Rocky River that each time a greater installation of pumps was investigated the development looked more favorable, and the pumping capacity is now double that originally authorized. If this example is typical of other locations the present estimates of the country's potential water power will have to be increased many times, and the strides made in the last few years in Europe lead to the conclusion that the development of the pumped-storage hydro-electric plant in the United States will go forward at a swift pace.

II.—Description of Plants

A brief description of the most important plants will illustrate the application of the principles discussed.

Mill Creek No. 2, Murray, Utah.—This small part-head plant, built in 1913, is the first American installation on which data have been found. The striking statement has been published* that the entire cost of the pumping features added to a high head development ($5 500), was less than the value of the additional saleable energy produced annually.

* *Engineering Record,* March 15, 1913.

TABLE 1.—LIST OF PLANTS.

Plant No.	Date of construction.	Location.	* Type.	Static Head, in Feet.		Reservoir elevation, in feet.	Present Net Reservoir Capacity, in Billion Cubic Feet.		Present Pumps.				Horse-Power Capacity of Present Turbines.		Reservoir with or without natural inflow.	Reference numbers in Bibliography.
									Largest.		Total.					
				Power.	Pumping.		Head-water.	Tail-water.	Horse-power.	Cubic feet per second.	Horse-power.	Cubic feet per second.	Largest.	Total.		
1	1882	Zurich, Switzerland (hydro-mechanical until 1891)	Pondage	510	510		0.0006		400		700					22
2	1885*	American hydro-mechanical	Pondage													No published data
3	1894	Luino, Switzerland (?)	Pondage						100		100				Without	26
4	1899	Choindez, Switzerland		1 970	1 970											22
5	"	Clus, Switzerland		1 030	1 030											22
6	1903	Ruppoldingen, Switzerland														26
7	1904	Olten-Aarburg, Switzerland	Pondage	1 030	1 030		0.0004		800		800				Without	22 and 26
8	1908	Brunnenmuhle (near Heidenheim), Germany	Pondage	330	330		0.0002		160		240				Without	22 and 26
9	1909	Schaffhausen, Switzerland	Pondage	530	530		0.003		1 000		2 000				Without	2, 22, and 26
10	"	Clenezzo (near Bergamo), Italy	Pondage	1 410	1 410				785		1 570				Without	22 and 26
11	"	S. El. d'Evian-Thonon-Annamasse, France	Pondage	1 310	1 310		0.0004		600		600				Without	26
12	1910	Chevenoz, Haute-Savoie, France	Pondage	1 310	1 310		0.0003		800		800					22
13	"	Stura di Viu, or Funghera, Turin, Italy	Pondage	500	500		0.002	0.002	4 000		4 000		4 000		Without	11, 22, and 26
14	"	Cornabbia, Italy	Pondage	80	80				5 800		5 800				Without	26
15	1913	Viverone, Novare, Italy	Pondage	490	490		0.01		4 500		9 750				With	11 and 22
16	"	Neckartzlingen, Germany	Pondage	410	410		0.006		320		540				Without	26
17	"	Mill Creek No. 2, Murray, Utah	Part head	1 030	140		0.0008		100	4	100	4	1 300	2 600	With	10
18	1915	Lake Fully, Valais, Switzerland	Part-head	5 410	460	6 990	0.1		570	7	570	7	3 000	12 000	With	4, 9, 13, and 20
19	1920	Fridingen, Germany	Pondage	570			0.001		620	7	1 860	21	750	2 250		26 and 28
20	"	Fully, Switzerland	Pondage	490	490				300		600					26

* Generator kilowatt.

TABLE 1.—(*Continued.*)

Plant No.	Date of construction.	Location.	Type.	Static Head, in Feet.		Reservoir elevation, in feet.	Present Net Reservoir Capacity, in Billion Cubic Feet.		Present Pumps.				Horse-Power Capacity of Present Turbines.		Reservoir with or without natural inflow.	Reference numbers in Bibliography.
									Largest.		Total.					
				Power.	Pumping.		Head-water.	Tail-water.	Horse-power.	Cubic feet per second.	Horse-power.	Cubic feet per second.	Largest.	Total.		
21	1922	Tubingen, Germany	Pondage	370	370	......	0.0002	......	190	3	......	4	290	290	Without	16, 17, 28, and 30
22	1923	Uberlingen, Germany	Storage	360	360	......	0.05	......	580	11	580	11	600	800	With	28
23	"	Schwarzenbach (near Forbach), Baden, Germany	Part-head	1 220	730	......	0.5	......	10 000	110	20 000	220	27 500	55 000	With	7, 25, 26, and 29
24	"	Munster, Alsace, France	Pondage	1 310	1 310	......	0.0006	0.0005	1 800	9	3 190	......	2 100	2 100	Without	21 and 26
25	" *	Ill Lake, Switzerland	Part-head	3 230	360	7 680	......	......	500	8	1 500	24	......	......	With	9
26	1924	Lake Girotte, or Belleville, Savoie, France	Storage	1 620	1 620	5 650	1.	......	4 500	16	9 000	32	......	......	With	11 and 23
27	1925	Reutlingen, Germany	Pondage	390	390	......	0.0014	0.0028	......	23	......	35	2 000	......	......	17 and 30
28	"	Waggital, or Rempen, Switzerland	Storage	800	800	2 950	5.	......	5 000	45	20 000	180	19 000	76 000	With	8, 12, 15, and 26
29	"	Zweribach, Germany	Pondage	1 640	1 640	......	0.0002	......	1 000	5	3 000	15	780	2 340	With	3 and 26
30	"	Tremorgio, Tessin, Switzerland	Storage	2 900	2 900	6 000	0.3	......	......	......	13 000	......	12 000	12 000	With	7, 15, 18, and 26
31	1926	Rhone Valley, Switzerland	Part-head	2 930	300	7 710	......	......	500	......	1 500	......	4 200†	8 400†	......	5
32	"	Kargegg, Germany	......	410	......	......	......	......	......	......	......	......	......	......	......	26
33	"	Konigstuhl (near Heidelberg), Germany	......	1 480	......	......	......	......	......	......	......	......	......	......	Without	26
34	" *	Walchen Lake, Bavaria, Germany	......	......	......	......	......	......	24 000	......	......	......	......	......	......	16 and 27a
35	1927	Niederwartha (near Dresden), Germany	Pondage	460	460	......	0.07	0.07	27 000	350	108 000	1 400	30 000	120 000	......	7, 12a, 24, and 25
36	"	Hemfurth, Prussia, Germany	Storage	130	130	......	......	......	8 400	480	16 800	960	......	33 000†	With	26
37	"	Hengstey, Ruhr, Germany	Pondage	520	520	......	0.04	0.04	34 000	420	102 000	1 260	45 000	180 000	With	12a, 23a, and 26
38	"	Mittweida, Saxony, Germany	......	......	......	......	......	......	......	......	......	......	......	......	......	25
39	"	Munich, Bavaria, Germany	......	......	......	......	......	......	......	......	......	......	......	......	......	25
40	"	Maen, Italy	Part-head	2 730	820	6 900	......	......	......	......	......	......	......	......	With	9
41	"	Luner Lake, Vorarlberg, Austria	Part-head	4 500	1 600	6 360	......	......	......	......	......	......	......	......	With	9 and 27a
42	"	Rocky River (near New Milford), Conn.	Storage	230	230	430	6.	......	8 100	250	16 200	500	33 300	33 300	With	1, 3a, 7a, 9a, 9b, and 12b

* Generator kilowatts. † Generator kilovolt-amperes.

The main development consists of a 20-ft. dam forming a reservoir with a capacity of 300 000 cu. ft., from which the water is led by a 2.5-ft. pipe line, 27 200 ft. long, to a power-house. Two 1 300-h.p. turbines are installed at this power-house for a head of 1 030 ft. One-half mile below the dam are a number of springs with a total flow of 3 to 4.5 sec-ft., and a 4-ft. dam has been built to collect their discharge. A 100-h.p. pump, delivering 3.5 sec-ft., and operated automatically by a float-switch, pumps water against a head of 140 ft. through a 12-in. pipe into the main pipe line on the hillside above.

The operation of this plant is not entirely on the pumped-storage principle because the pumping is controlled by a float-switch and is not exclusively off-peak. At the Lake Fully plant, in Switzerland, also, the water from the pump flows up or down the penstock, depending on the load on the turbines.

Tübingen, Germany.—The reservoir for this plant, which was built in 1922, is of reinforced concrete. It is one of three which are smaller than any of the others, containing only 200 000 cu. ft., or 2 000 000 gal. The pool is 20 ft. deep and only 144 by 82 ft. in plan. Rocky River has a capacity 30 000 times as great. The reservoir is set upon a hill 370 ft. above the Neckar River, where the power-house is located, and is filled in 16 to 20 hours through a 15-in. pipe, 2 000 ft. long. In the power-house is a unit typical of present European practice even for the largest plants. It consists of a dynamo with a horizontal shaft, on one end of which is a turbine, and on the other end, a pump, the dynamo acting as a motor or as a generator as occasion demands. At Tübingen, the turbine is rated at 290 h.p. and the pump at 190 h.p., delivering 3 sec-ft. Additional pumping capacity is supplied by a pump delivering 0.9 sec-ft.

The smallness of this reservoir, taken in conjunction with those at Brunnenmühle and Zweribach, which are equally small, seems to indicate that while economic conditions are not the same in America as in Europe, it might pay some of the smaller companies in this country, when they need a few hundred kilowatts of firm capacity, to look around their hills and brooks to see whether there is not some particularly favorable location worth investigating for pumped-storage.

Schwarzenbach, near Forbach, Baden, Germany.—The adaptation of an existing hydro-electric plant to secure additional firm power makes this plant unusual. Previous to 1923 the Murg plant had been built for a head of 490 ft., with a forebay of a capacity of 10 000 000 cu. ft., an 11-ft. tunnel, 18 000 ft. long, and two penstocks, 1 000 ft. long, supplying five 7 000-h.p. turbines. For the Schwarzenbach plant these penstocks have been tapped at the bottom and water is drawn from them and pumped to a reservoir 730 ft. above the forebay of the Murg plant, or 1 220 ft. above the old tail-water level. This reservoir has a present capacity of 500 000 000 cu. ft. (600 000 000 cu. ft. in the future) for a drawdown of 130 ft. and is filled through two 6-ft. penstocks, 2 600 ft. long, by two combination units, each of which has two pumps connected by gears to the main shaft. The two pumps of each unit together are rated at 10 000 h.p. and deliver 110 cu. ft. per sec. with an efficiency of 85%, the highest in Europe, while the turbines are of 27 000 h.p.

TABLE 2.—LIST OF PROPOSED EUROPEAN PLANTS.

Plant No.	Date of construction.	Location.	Type.	Static Head, in Feet.		Reservoir elevation, in feet.	Ultimate Net Reservoir Capacity, in Billion Cubic Feet.		Ultimate Pumps.				Horse-Power Capacity of Ultimate Turbines.		Reservoir with or without natural inflow.	Reference numbers in Bibliography.
									Largest.		Total.					
				Power.	Pumping.		Head-water.	Tail-water.	Horse-power.	Cubic feet per second.	Horse-power.	Cubic feet per second.	Largest.	Total.		
1	1914	Colmar, Haut-Rhin, France		330	330						25 000†	1 270			with	22
2	1927	Chemnitz-Zwickau Region, Saxony, Germany														26
3	"	Leipzig, Saxony, Germany														26
4	"	Dresden, Germany														26
5	"	Struden, Germany		510	510		1.							107 000	with	19 and 26
6	"	Our River (near Stolzenberg), the Eifel, Germany	Storage				28.	0.3			250 000*			600 000†	with	18, 19, and 26
7	"	Siemens-Schuckert System, Germany	Pondage	820	820		0.004		6 550	56	13 100	112	10 900	43 600	without	27
8	"	Hagen, Westphalia, Germany	Pondage	570	570									12 000†		26 and 31
9	"	Laacher Lake (near Namedy), Essen, Germany				900									with	31
10	"	Mort Lake, (near Grenoble), France	Storage			2 050	0.2		4 800					12 500	with	6 and 14

* Motor-kilowatts.

† Generator-kilowatts.

Tremorgio, Tessin, Switzerland.—This plant is an excellent example of the addition of pumping equipment at an existing development to add to its value. Its reservoir was originally formed for pure storage by damming a natural lake and raising its surface to Elevation 6 000. Later, a 5-ft. tunnel, 600 ft. long, was driven to tap the lake and secure a capacity of 300 000 000 cu. ft. for a draw-down of 90 ft. From the tunnel a penstock 5 100 ft. long and 2.3 ft. in diameter, with provision for a second, leads to a 12 000-h.p. turbine under 2 900-ft. head in the power-house which is located on the Tessin River. This head is the highest in a full-head development. In 1926 two pumps with a total capacity of 13 000 h.p. were added, making it a pumped-storage plant. An unusual feature of this development is the tapping of the penstock to supply water to several towns.

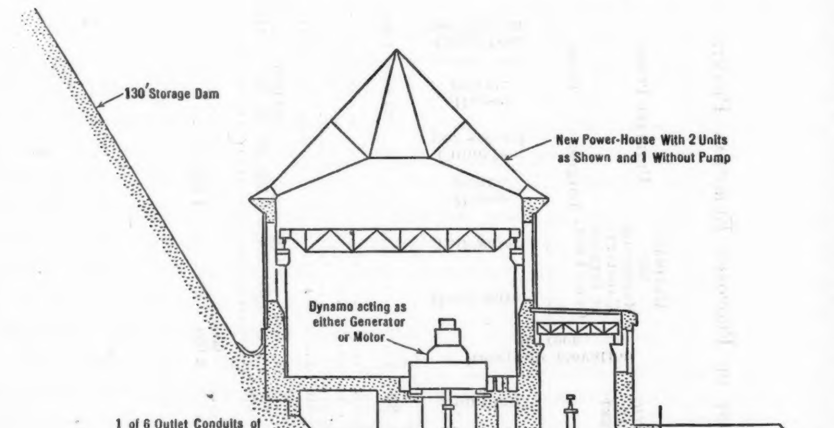

FIG. 4.—ADAPTATION OF STORAGE AND POWER PLANT TO THE PUMPED-STORAGE PRINCIPLE.

Hemfurth, Prussia, Germany.—This is a unique addition of pumping and power equipment to an existing development. The original construction consisted of a 130-ft. storage dam with 15 000 kw. installed in the power-house and six outlet conduits discharging through needle-valves into a stilling pool. This has been transformed into a pumped-storage plant (Fig. 4) by tapping two of the conduits and connecting each of them by branches to a turbine-dynamo-pump unit in a second power-house. A dam has been built down stream to form a pool from which the pumps can draw water, with a fluctuation of 7 ft. in the water level.

The pumps in this development have the greatest discharge of any so far built (480 sec-ft.), although their rating of 8 400 h.p. does not approach a record. The turbines are of 7 900 h.p., and the generators are rated at 6 000 kw. In addition to the two combination units there is a third unit with the pump omitted. The units are unusual in the fact that the shaft is vertical. If the plant had been designed originally as a pumped-storage plant it is possible that it could have been designed for greater hydraulic efficiency, but as an example of the application of the pumped-storage principle to a plant already constructed, it is highly interesting.

Niederwartha, near Dresden, Germany.—This is the type of plant which, in the writer's opinion, is likely to become common near metropolitan load centers in this country wherever there are hills. More than a dozen possible developments are already under consideration. The Niederwartha plant improves the operating conditions of the Dresden steam plants with which it is connected, by smoothing off the peaks and filling in the valleys of the load curve and by standing as a reserve plant available almost instantly. Two reservoirs have been constructed (an upper and a lower), each of 70 000 000 cu. ft. capacity. Water is drawn from the Elbe River near-by to fill the lower reservoir, from which it is pumped into the upper reservoir, 460 ft. above, by four 27 000-h.p. pumps. These are part of combination turbine-dynamo-pump units in which the turbines are rated at 30 000 h.p. and the dynamos at 15 000 kw. The ultimate installation will be six such units, and three pipe lines have been built, 9.2 ft. in diameter and 3 000 ft. long. Each pump discharges 350 sec-ft.

Hengstey, Ruhr, Germany.—The 34 000-h.p. pumps at this plant are the most powerful yet built, and their rating is four times that of any in the United States. The 45 000-h.p. turbines at Hengstey are the most powerful in Germany, and probably in Europe; the turbine capacity of the plant, 180 000 h.p., is the largest total; and the annual output is 120 000 000 kw-hr.—yet here power is only a by-product of domestic water supply.

A settling basin was required on the Ruhr River and after consideration of various plans it was found that the capacity of the basin could economically be tripled if a pumped-storage plant of the pondage type were built. On a bluff on one bank of the river a reservoir with a net capacity of 40 000 000 cu. ft. for 45-ft. draw-down is being constructed by building an oval dike, and this reservoir is being connected with the power-house on the river by four 8.2-ft. penstocks, 1 000 ft. long. In the power-house are three combination turbine-dynamo-pump units and a fourth unit with the pump omitted. The head is 520 ft. and each pump discharges at the rate of 420 sec-ft. The sedimentation basin, which has a gross capacity of 100 000 000 cu. ft., is formed by a dam at which there is a run-of-stream plant developing a 15-ft. head. The lowering of the water level in the basin due to the draft to fill the upper reservoir, is 2.3 ft.

Rocky River, near New Milford, Conn.—This development has four outstanding features: (1) It is the first modern plant in America; (2) its reservoir is the largest of all, having a net capacity of 6 000 000 000 cu. ft. for a 30-ft. draw-down with an ultimate capacity of 7 000 000 000 cu. ft. when raised

5 ft.; (3) its elevation of 430 ft. above sea level is the lowest; and (4) its pumps have the highest efficiency, 87% being guaranteed. The lowness of the elevation of the reservoir indicates that pumped-storage is not limited to mountainous country where there is a large output from each cubic foot of water stored. An indicated in Fig. 2, Rocky River is a small tributary of the Housatonic River entering a mile above New Milford. Fig. 5* is a photograph made during the filling of the reservoir. The dam is of earth. The pumps are at the right end of the power-house. The water is led from the reservoir by a canal 3 200 ft. long and 45 ft. deep, and a 15-ft. pipe line, 1 900 ft. long, to a power-house which contains one 24 000-kw. generator and two 8 100-h.p. pumps, each delivering 250 sec-ft. Provision has been made for a second generator. The head is 230 ft. and the drainage area, 40 sq. miles, while the drainage area of the Housatonic at the power-house is 1 037 sq. miles. With two run-of-stream plants in the system the addition of firm capacity to the system by the building of the Rocky River plant will be 40 000 kw. and the additional output will be 80 000 000 kw-hr. Its regulating effect on the river will also make possible otherwise uneconomical projects.

Acknowledgments

Thanks are due Mr. L. D. Gilfillan for many discussions with the writer, and to him and to Mr. M. S. Weil for valuable criticism. The writer wishes especially to express his deep appreciation of the criticism given by Joel D. Justin, M. Am. Soc. C. E., which has been invaluable.

The writer would appreciate any additional data to be presented in the discussion, particularly where statements of extremes or uniqueness have been based on data presented herein, which are, of course, incomplete, although indicative of the trends.

APPENDIX

Methods of Operation

Power is produced from a plant of the storage type like Rocky River by the following six methods. It must be remembered, however, that only average monthly flows are considered in this discussion, variations within the months being ignored.

1.—Pondage.—Starting with full reservoir at the end of the spring freshet season, the plant is held in reserve and is idle except as explained in Sections 3 to 6. The plants of the combined steam and hydro-electric system are to be operated so as to obtain the maximum possible firm capacity from the water available. This capacity has been determined by a trial-and-error computation, in which it has been discovered that when the average monthly flow of the main river at the power-house of the pumped-storage plant has fallen to a certain quantity, it is necessary to operate the generator in the pumped-storage plant in order to maintain the firm capacity of the hydro-electric system. The unit, therefore, is operated, but all water released during the

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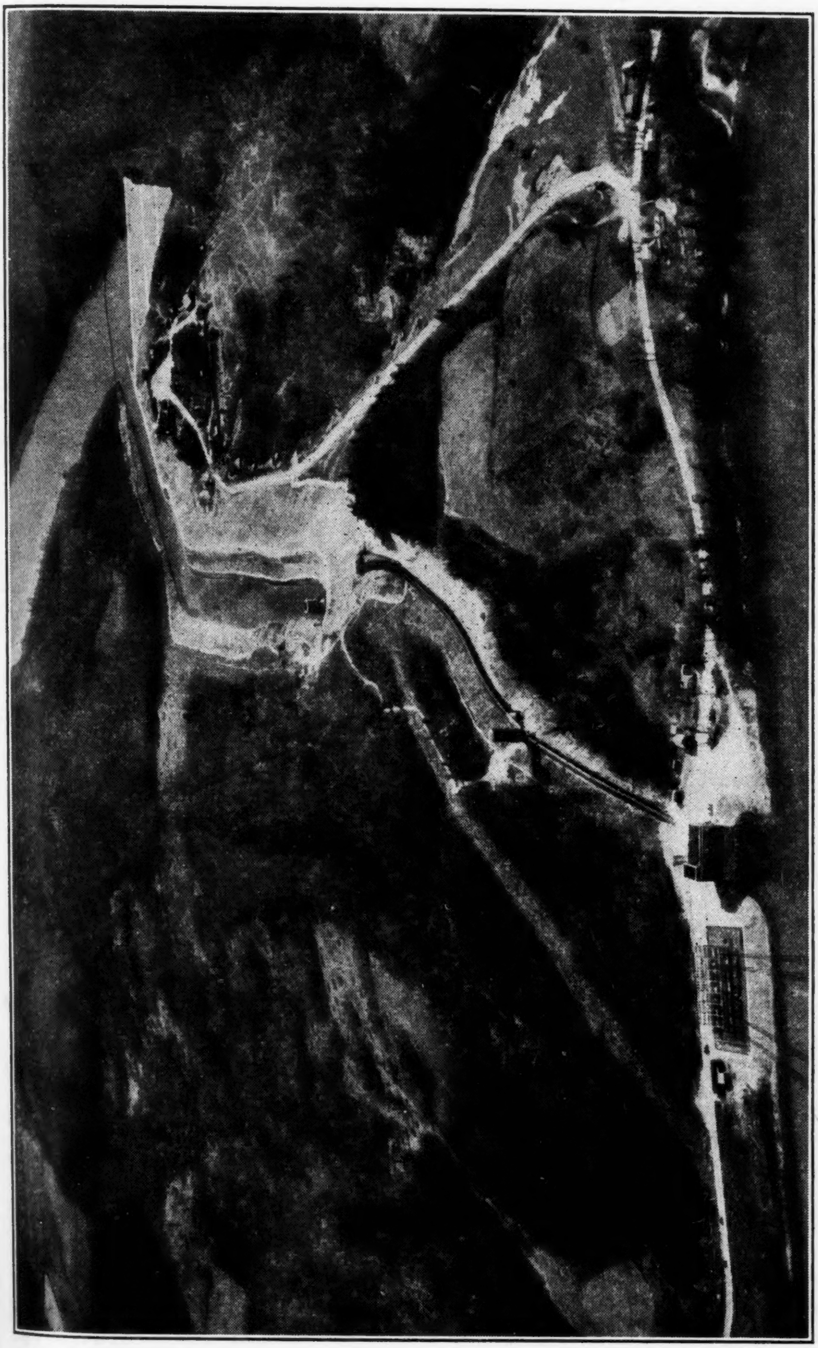

FIG. 5.—ROCKY RIVER DEVELOPMENT FROM A RETOUCHED PHOTOGRAPH MADE DURING THE FILLING OF THE RESERVOIR.

on-peak hours of the day is replaced by water pumped up into the reservoir by steam-generated power during the off-peak hours of the night and on Sunday. The firm hydro-electric capacity is thus maintained without any net depletion of the reservoir and the flow of the river is unaffected by the storage reservoir.

2.—Flow Regulation.—When the flow of the river falls still lower, the firm capacity of the hydro-electric plants of the system is maintained at the amount determined by the computation mentioned previously, by releasing water from the reservoir to be added to the flow of the river for the benefit of the plants down stream as well as to produce power at the pumped-storage plant. This water is in addition to that which is released during the day to be pumped up again during the night (see Method 1).

3.—Secondary Energy from Natural Inflow.—As the flow of the main river increases after the low-flow period the flow-regulation operation and the pondage operation of the plant are successively stopped. When the flow increases so that the pumps can draw from the river without decreasing the firm capacity by withholding water from the down-stream plants, the reservoir is refilled by pumping. Finally, when the reservoir is full the natural inflow is used to produce secondary energy and prevent the reservoir from overflowing; but otherwise the plant is idle, except as indicated by Methods 4, 5, and 6, until the next decrease in the flow of the main river, when the cycle is repeated.

In tabular form this fundamental cycle is as follows:

Flow of Main River Decreasing:

(*a*) Full reservoir.
(*b*) Secondary energy from natural inflow into full reservoir.
(*c*) Pondage operation with full reservoir.
(*d*) Pondage and flow-regulation operation drawing down reservoir with increasing rapidity.

Flow of Main River Increasing:

(*d*) (Continued) Pondage and flow-regulation operation drawing down reservoir with decreasing rapidity.
(*e*) Pondage operation with reservoir drawn down.
(*f*) Plant idle with reservoir drawn down.
(*g*) Refilling reservoir.
(*a*) Full reservoir again.

This cycle is reversed if at any point the tendency of the stream flow changes; that is, if in the idle period, (*f*), the flow decreases instead of increasing, the pondage operation of Period (*e*) is resumed, followed by Period (*d*) until the flow again increases, when the operation goes forward on the cycle through Periods (*e*), (*f*), and (*g*).

The maximum firm capacity of the hydro-electric system is maintained by operation on this cycle, but under certain conditions this operation of the plant may be supplemented as described under Methods 4, 5, and 6.

4.—Secondary Energy with Surplus Hydro-Electric Energy.—If the plant is operated on the fundamental cycle, Sections (1) to (3), there are some parts of Period (*b*) when the reservoir is full and the plant is producing secondary energy from the natural inflow only, while water is being wasted over the dams

of run-of-stream plants at night and on Sunday, the units being only partly loaded. Under these conditions coal can be saved at steam plants by drawing on the reservoir during the peak hours and refilling it by pumping with surplus hydro-electric energy during the off-peak hours. This secondary energy from water which is replaced by pumping is in addition to the secondary energy from the natural inflow when the reservoir is full, as described under Method 3.

5.—Secondary Energy Instead of High-Cost Steam.—Again, if the plant is operated on the fundamental cycle, there may be times in Period (*b*) (while the reservoir is full) when for a few hours it may be necessary to start and operate another boiler or turbine in a steam plant. The unit cost of the energy produced is very high and, as shown in the following example, it may pay to operate the pumped-storage plant for short-time storage of secondary energy.

Assume the on-peak cost of steam-generated power under the condition described to be 8 mills and the off-peak cost, 4 mills, both at the station bus. Alternative means of supplying an additional load of short duration, with the run-of-stream plants already using all the available water and one steam turbine fully loaded, are to start either a second steam turbine or the pumped-storage plant, replacing the water used shortly afterward. Energy from either plant is delivered at the same sub-station, high-tension bus, at which point the costs may be compared.

Example.—The cost of steam-generated energy may be analyzed as follows:

Efficiencies:	Percentage.
Steam station step-up transformers..................	98
Relatively short transmission line to load center.........	95
Resultant..............................	93

The cost at the steam station switchboard is 8 mills per kw-hr., and at the sub-station, high-tension bus, 8.6 mills per kw-hr. delivered. This cost per kilowatt-hour delivered compares with the cost of pumped-storage hydro-electric energy as follows:

Efficiencies:	Percentage.
Steam station step-up transformers..................	98
Relatively long transmission line to pumped-storage plant.	90
Hydro-electric plant step-down transformers...........	98
Pump motor fully loaded........................	96
Pump fully loaded..............................	87
Penstock to reservoir lightly loaded..................	99
After an interval of time between off-peak and on-peak:	
Penstock from reservoir partly loaded................	98
Turbine partly loaded...........................	89
Generator partly loaded.........................	96
Hydro-electric plant step-up transformers.............	98
Relatively short transmission line to load center.........	95
Resultant..............................	56

The cost at the steam-station switchboard is 4 mills per kw-hr., and at the sub-station, high-tension bus, 7.2 mills per kw-hr. delivered. The saving from pumped-storage instead of steam-generated energy is 1.4 mills per kw-hr., or 16 per cent. The ratio of the resultant efficiencies is 0.60.

The extent of this use of the pumped-storage plant changes with the system load, available units, cost of energy, and stream flow, and it is ignored in the financial statement of the value of the plant.

6.—Break-Down Power.—In an emergency the hydro-electric unit, if not already generating power, can be placed in operation in a few minutes. The difference, however, between the pumped-storage plant where the tail-race is a pool, and any other hydro-electric plant is that this emergency use is not dependent on the stream-flow conditions, for the water can be retained in the pool and pumped back into the reservoir, while in other plants after the water is once released it is gone. This pumping, and pumping for pondage, are limited to the flow of the main river if there is no pool forming the tail-race.

For a plant to have definite reserve value which can be included in the financial statement there must be a big river or pool forming the tail-race and balanced generating and pumping equipment provided for operation outside the fundamental cycle.

In case there is no after-bay and the pumping is limited to the flow in the main river, the production of secondary energy at any time except when the reservoir is full, as discussed under Methods 4, 5, and 6, may jeopardize the filling of the reservoir before the next low-water period and may lower the firm capacity of the hydro-electric system.

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(2) **Electrical World,** September 16, 1911; (3) December 19, 1925; (3*a*) July 31, 1926; (4) November 6, 1926; (5) February 19, 1927; (6) June 25, 1927; (7) July 16, 1927; and (7*a*) May 12, 1928.

(8) **Engineering News-Record,** August 28, 1924, September 9, 1926, and June 14, 1927; (9) November 3, 1927; (9*a*) January 12, 1928; and (9*b*) February 16, 1928.

(10) **Engineering Record,** March 15, 1913.

(11) **Power,** November 22, 1921; (12) May 5, 1925, and January 25, 1927; (12*a*) February 28, 1928; and (12*b*) May 28, 1928.

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(23*a*) **Das Gas- und Wasserfach,** January 28, 1928.

(24) **Elektrizitätswirtschaft,** March, 1927, Section 2; and (25) September, 1927.

(26) **Elektrotechnische Zeitschrift,** June 30, 1927.

(27) **Siemens Zeitschrift,** June, 1927.

(27*a*) **Schweizerische Wasserwirtschaft,** March, 1927.

(28) **Zeitschrift des Vereines deutscher Ingenieure,** November 8, 1924; (29) November 15, 1924; (30) December 27, 1924; and (31) April 16, 1927.

Therefore, if the use of the pumped-storage plant changes with the season [illegible] available units [illegible] and [illegible] ignored in the [illegible] agreement of the value of the plant.

[illegible] hydro-electric unit [illegible] placed in operation in a few minutes. The [illegible] between the pumped-storage plant [illegible] that the [illegible] the water can be retained in the [illegible] pumped height [illegible] while the water [illegible] may be included in the [illegible] inside [illegible]

[illegible]

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

ECONOMIC COMPARISONS OF VARIOUS TYPES OF ROAD SURFACING

A SYMPOSIUM*

NOTE: Discussion on this Symposium will be closed in February, 1929.

* Presented at the meeting of the Highway Division, New York, N. Y., January 19, 1928.

ECONOMIC COMPARISONS OF ROAD SURFACES—NEW HAMPSHIRE

By F. E. Everett,* M. Am. Soc. C. E.

In New Hampshire at the close of the 1927 season the total mileage of improved highways on the State Aid and Trunk-Line Systems was approximately 2 250, all of which has been built since the State Highway Department was established by a Legislative Act in 1905.

The various road surfaces on the two systems vary from common gravel to the highest type of modern reinforced concrete. Experience in New Hampshire indicates that when traffic exceeds approximately 500 vehicles per day, ordinary untreated gravel and similar surfaces cannot be maintained economically for satisfactory service. Although plain gravel was found to be acceptable during the early period of development (prior to 1916), the increase in traffic finally made it impossible to provide proper service for gravel roads on the principal routes, without the application of a bituminous surface treatment. Consequently, between 1916 and 1921, gravel roads to the extent of 571.2 miles were treated with tar and asphaltic compounds. Even in 1928 surface-treated gravel has continued to be the principal road surface, with more than 850 miles of this type on the Trunk-Line System and nearly 350 miles on the State Aid roads. The other types on the Trunk-Line System include 150 miles of bituminous macadam, 75 miles of modified asphalt or asphaltic concrete, and 30 miles of reinforced cement concrete (15 miles of this having been built during 1927). The 1928 State Aid Road System consists chiefly of improved sections of scattered and unconnected highways mostly of plain gravel, of which there is a total of approximately 800 miles.

Due to increased traffic and the necessity of reconstructing worn-out surfaces, maintenance and construction costs on gravel roads amounted to 42.9% of the total highway expenditures during the 5-year period, 1921 to 1925, inclusive. In 1925 it increased to 45.3% of which approximately two-thirds was for maintenance alone. Most of the maintenance costs can be attributed to gravel surfaces which become increasingly expensive as traffic increases. Therefore, this type cannot be termed an economic road surface under heavy traffic conditions. However, when such a road is properly treated with tar or asphaltic compound, it will answer in many cases the purpose of a more expensive surface. During the 1927 flood many miles of this surface withstood the swift waters, boulders, and other débris passing over it.

As a basis for a 10-year plan of improvement in New Hampshire, the Trunk-Line System has been classified in three groups, designated as major, medium, and minor routes, according to their average daily present, and estimated future, traffic. Major sections carry 1 500 or more vehicles daily; medium routes, from 500 to 1 500, daily; and minor routes, less than 500, daily. This classification has been made on the basis of the observed 1926 traffic. Also, the traffic in 1931 and 1936 has been estimated.

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In order to obtain construction costs for an economic comparison of the various types of road surfaces in New Hampshire, several sections of the Trunk-Line System are indicated in Table 1. However, since all the projects recorded average approximately 1 mile in length, the unit cost is, of course, somewhat greater than for longer projects. All the roads are constructed under Federal Aid specifications and have a width of 18 ft. with 3-ft. shoulders.

TABLE 1.—CONSTRUCTION COSTS IN NEW HAMPSHIRE.

Type.	Number of projects.	Total miles.	Average cost per mile.
Reinforced cement concrete	12	13.89	$41 132.82
Asphaltic concrete	24	39.40	22 549.09
Bituminous macadam	84	73.78	26 057.09
Surface-treated gravel	50	39.76	20 531.36

In order to get a fair comparison of the yearly maintenance costs for the same types, several new construction projects are listed in Table 2, with maintenance costs for the first four years after completion. Data are not available for comparisons for longer periods. The first section of reinforced cement concrete built is five years old, with maintenance data for four years. Costs for surface-treated gravel and bituminous macadam are for longer periods; however, they do not vary greatly from the data of Table 2, except that bituminous macadam requires a seal coat about once in five years and that surface-treated gravel needs a general re-shaping, scarifying, and re-surfacing at least once during the same period.

TABLE 2.—MAINTENANCE COSTS IN NEW HAMPSHIRE.

Type.	Classification of traffic sections.	Cars per day.	Trucks per day.	Years.	Cost of maintenance for period as noted.	Cost of maintenance per year.
Reinforced cement concrete	Major	2 000	110	4	$ 151.00	$ 37.75
Asphaltic concrete	Major	4 000	230	4	292.00	74.00
Bituminous macadam	Medium	1 200	100	4	263.00	65.75
Surface-treated gravel	Medium	1 000	100	4	2 432.00	608.00

The maintenance costs in Table 2 are considerably less than the average throughout the State, especially for surface-treated gravel, as they deal with new projects only. The resulting total costs are given in Table 3.

For comparison over a period of ten years, it is necessary to add to the bituminous macadam maintenance the cost of an extra seal coat. Also, reconstruction cost must be added to the surface-treated gravel road within the same period. The results are shown in Columns (5) and (6) of Table 3.

Asphaltic concrete appears to be the most economical pavement for the 10-year period. However, as this type has been confined for the most part to

reconstruction projects where there is already a prepared base, its cost cannot with fairness be compared with the others. This leaves surface-treated gravel as the most economical, with bituminous macadam costing only $1 000 per mile over the 10-year period. However, it should be noted that the comparisons on these surfaces were derived from the 4-year period on medium traffic routes. Bituminous macadam as built by New Hampshire specifications has a 3-in. bituminous top laid on a 3-in. macadam, or crushed gravel, base. This cannot be classed as a heavy duty or major traffic road surface, such as is used in the reinforced cement concrete construction. Specifications for reinforced cement concrete call for a center thickness of 6 in., with 9 in. at the shoulders. Experience does not show just how long this construction will last; for the bituminous type it is ten years, after which it becomes necessary to apply a new surface. Mistakes have been made in the past by building too light surfaces on some of the main trunk-line roads. This was due, first, to the initial cost; and, second, to not anticipating the future traffic. Sections of the Daniel Webster Highway, originally built with water-bound macadam and gravel, were entirely rebuilt with bituminous macadam and in no place more than 6 in. in thickness. Had the present traffic been foreseen (1912-20) these same sections would have been built with a much heavier type of construction. To-day, with the advantage of the traffic survey, such errors in constructing for the present as well as for the future can be avoided.

TABLE 3.—Total Costs.

Type. (1)	Cost per mile. (2)	Maintenance per mile per year. (3)	Yearly interest on investment at 5 per cent. (4)	Maintenance and interest cost, 10 years. (5)	Total cost, 10-year period. (6)
Reinforced concrete..........	$41 132.82	$37.75	$2 056.64	$20 943.90	$62 076.72
Asphaltic concrete..........	22 549.09	74.00	1 127.45	12 014.50	34 563.59
Bituminous macadam........	26 057.09	65.75 + $80.00*	1 302.85	14 486.60	40 543.69
Surface-treated gravel........	20 531.36	608.03 + 300.00†	1 026.56	19 345.90	39 877.26

* Seal coat necessary once in five years, $800 per mile.
† Reconstruction necessary once in five years, $3 000 per mile.

A bituminous macadam with a heavier base to compare with modern reinforced cement concrete construction, would cost much more, of course, than the particular projects tabulated. Again, by using for analysis data derived from projects built in 1927, the average cost per mile for reinforced cement concrete construction would have been reduced to $35 957.42 and that for bituminous macadam, advanced to $30 446.37.

The decrease for the reinforced cement concrete is due, first, to building larger projects, thus reducing the overhead; and, second, to the scientific design of the mixture using the water-ratio method which was reflected in the contractor's prices. The increase in cost of the bituminous macadam construction is due to incorporating a heavier sub-base in the road.

Cost comparisons between the reinforced cement concrete construction and bituminous macadam in 1927, given in Table 4, show a difference of approximately $7 000 per mile for the 10-year period.

TABLE 4.

Type.	Initial.	Interest for 10 years at 5 per cent.	Maintenance, 10 years.	Total.
Reinforced concrete	$35 957.42	$17 978.71	$377.50	$54 313.63
Bituminous macadam	30 446.27	15 223.13	1 457.50	47 126.90

Lack of sufficient data on construction and maintenance costs of different types of pavements for the same period, traffic, etc., makes it impossible to give proof of the economic advantage of any one type of pavement over another. However, experience in New Hampshire leads to the belief that bituminous macadam is a 10-year pavement. It is hoped and expected that the modern reinforced cement concrete pavement will have a minimum life of fifteen years, judging from the known experiences of other States. Thus, the State will continue to construct reinforced cement concrete on its major traffic routes; bituminous macadam on its medium traffic routes; and surface-treated gravel on minor traffic routes.

Therefore, the writer's conclusion is that the best type of road surface varies directly with the amount of traffic. Consequently, New Hampshire will strive to follow closely the construction and reconstruction program as outlined by the results of traffic surveys and endeavor to construct road surfaces that will meet the present and forecasted future demands.

ECONOMIC COMPARISONS OF ROAD SURFACES—WISCONSIN

By H. J. Kuelling,* M. Am. Soc. C. E.

In Wisconsin there are about 3 000 miles of concrete pavement and several times as much gravel surfacing. This study, therefore, is limited to these two types that are most prevalent. Unfortunately, no particular tests have been made for tire wear on surface-treated gravel roads. Hence the data herein given will not apply to other surface-treated roads as regards tire wear and gasoline consumption.

In this economic study it will be noted that some of the indirect costs are the predominating features. In other words, the primary costs, in so far as these two types are concerned, mean very little.

The builder, whether it be in highway work, or other lines, cannot go beyond his credit or the funds available. Much as he may desire a wide, smooth pavement extending through the highways and byways of his domain, he often must content himself with a much narrower and poorer quality of road. In other words, while it may be easy to show that such and such a pavement is cheaper in the long run than a gravel road, so likewise it is easy to demonstrate that the friendly gravel road is more economic than no surface at all.

The road movement, even from an economic standpoint, had to begin as a babe, crawl awhile, walk awhile, and now prepare to run awhile, with no knowing what speed it may eventually attain. Just as a merchant would be unwise to put in a stock of goods that was beyond the customers' capacity to buy, so paving the roads beyond the needs of the users would be wrong. Undoubtedly, the customers or users of highways have advanced and are now ahead of the builders. It will be a considerable time before the users are overtaken, or, in other words, before a system of roads is built to suit their demands, if not their needs.

Transportation, the problem of the ages, was never as acute as it is to-day. There never has been as much to transport as now, not only in total tonnage per capita, but in total miles traveled per capita. In fact, it is extremely doubtful whether any one can picture the transportation of the future. In 1914, officials were accused of extravagance in building county highways 18 ft. wide. Far better would it have been if they had been accused of stinginess. To-day, 20 ft. is the standard paving width with 30 ft., or even 40 ft., in special cases. No doubt in another decade this will seem niggardly and lacking in foresight. Undoubtedly, the road of the future will be wider and smoother than at the present. The users will demand the right to go farther and faster, carry bigger loads, have luxuries such as lights, be free from stops, and have better policing and greater safety.

This does not mean that gravel roads should not be constructed. Without question they will continue to occupy a very important place in the highway

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development, even if it is not economical to use them in comparison with some higher type. In Tables 5 to 8 are given the summaries of computations regarding the economics of Wisconsin highways. The costs of operation are taken from the tests made by T. R. Agg, M. Am. Soc. C. E., while the tire costs are taken from tests made by Mr. H. J. Dana, of the State College of Washington. Some statistics are also quoted from the report of the Fifth Annual Meeting of the Highway Research Board (December 3 and 4, 1925), which indicate that tire wear on macadam roads may be from two to seventeen times as much as on concrete and, on very poor macadam, as much as fifty-six times. On a surface-treated road, it was found that a certain quantity of oil was absorbed by the tire offsetting part of the wear and making it more difficult to obtain a true or actual loss of rubber. In building up Tables 5 to 8 certain assumptions, of course, are necessary. In one case, Tables 5 and 6, traffic density of 500 vehicles per day for the entire year was considered as increasing with a lessening degree until at the end of 20 years it had reached a constant rate of 1 500 vehicles per day. The actual tonnage was then computed, assuming that the average vehicle weighed $1\frac{1}{2}$ tons. In view of the increasing number of buses, trucks, etc., this is not believed to be extreme. In *Bulletin No. 69* of the Iowa State College, it is stated that the maintenance cost on the best type of graveled road is,

$$\frac{500 + A\,T}{3\ 000} \quad \text{.............................(1)}$$

in which, $A\,T$ is the annual tonnage. This assumes a constant maintenance charge of $500 to take care of the regular patrol, much of which is occasioned by bad weather, increasing with an increase in the volume of traffic. Using Equation (1), the maintenance charge for the various years was computed.

In computing the maintenance on concrete, this same Iowa report gave the amount as $\frac{80 + A\,T}{6\ 000}$. This seemed to produce results less than the Wisconsin experience, and the formula was changed to,

$$\frac{150 + A\,T}{6\ 000} \quad \text{.............................(2)}$$

The annual maintenance was computed from Equations (1) and (2) and the difference determined. It was then assumed that this saving could be set aside in a fund and draw $5\frac{1}{2}\%$ interest, compounded annually. This rate is probably high. If all these savings in maintenance were added together, with interest, the amazing sum of $14 788 would be credited in favor of the paved road (Table 5).

In comparing the original costs of construction, the items of grading and structures were omitted as they would be the same in either case. It was assumed that the gravel surfacing would cost $5 000 per mile (2 500 cu. yd. at $2 per cu. yd.). For every 5-year period, 1 000 cu. yd. of material would be added. If the original cost drew interest at the rate of $4\frac{1}{2}\%$, compounded annually, the total capital charged against the road at the end of 20 years would be $20 998. Deducting the assumed salvage value of the road, the net cost would be $18 998 (Table 5).

TABLE 5.—MAINTENANCE AND CONSTRUCTION COSTS PER MILE OF ROAD FOR 500 TO 1 500 VEHICLES PER DAY.

Year.	Estimated traffic per day.	Annual traffic, in thousands of tons.	Maintenance Cost. Gravel: $M = 500 + \frac{AT}{3\ 000}$.	Maintenance Cost. Concrete: $M = 150 + \frac{AT}{6\ 000}$.	Difference in maintenance cost.	Interest at 5½ per cent.	Accumulated savings.	Construction Costs. Gravel. Initial and reconstruction costs.	Construction Costs. Gravel. Interest at 4½ per cent.	Construction Costs. Gravel. Accumulated costs.	Construction Costs. Concrete. Interest at 4½ per cent.	Construction Costs. Concrete. Accumulated costs.
1	500	274	$591	$196	$395	....	$395	$5 000	....	$5 000	......	$24 000*
2	570	312	604	202	402	$22	819	......	$225	5 225	$1 080	25 080
3	630	345	615	207	408	45	1 272	......	235	5 460	1 130	26 210
4	700	384	628	214	414	70	1 756	......	245	5 705	1 180	27 390
5	770	422	641	220	421	97	2 274	2 000	256	7 961	1 230	28 620
6	835	457	652	226	426	125	2 825	......	358	8 319	1 290	29 910
7	890	487	662	231	431	155	3 411	......	374	8 693	1 340	31 250
8	960	526	675	238	437	188	4 036	......	391	9 084	1 410	32 660
9	1 020	559	686	243	443	222	4 701	......	408	9 492	1 470	34 130
10	1 080	591	697	249	448	258	5 407	2 000	427	11 919	1 540	35 670
11	1 140	625	708	254	454	298	6 159	......	536	12 455	1 600	37 270
12	1 195	655	718	259	459	338	6 956	......	560	13 015	1 680	38 950
13	1 250	685	728	264	464	383	7 803	......	586	13 601	1 750	40 700
14	1 300	712	737	269	468	429	8 700	......	612	14 213	1 830	42 530
15	1 350	740	747	273	474	478	9 652	2 000	640	16 853	1 910	44 440
16	1 385	759	753	276	477	531	10 660	......	759	17 612	2 000	46 440
17	1 430	784	761	281	480	586	11 726	......	793	18 405	2 090	48 530
18	1 460	800	767	284	483	640	12 849	......	828	19 233	2 190	50 720
19	1 480	816	772	286	486	707	13 556	......	865	20 098	2 280	53 000
20	1 500	822	774	287	487	745	14 788	......	900	20 998	2 380	55 380
..	......	....	....	....	......	....	......	Salvage value		2 000	......	15 000
..	......	....	....	....	$8 957	....	......	......	....	$18 998	......	$40 380
..	......	....	....	....	......	....	......	......	....	....	......	18 998
..	......	....	....	....	......	....	......	......	....	....	......	$21 382

* Initial cost.

In a similar manner it was assumed that a 20-ft. slab of pavement would cost $24 000 per mile and, at the end of a 20-year period, the net cost would be $40 380. This gives the gravel road an advantage of $21 382 based on these assumptions.

Tire expense is the subject of *Bulletin No. 17* of the State College of Washington which shows the average wear per tire per 1 000 miles to be 0.372 lb. This rubber is worth about $10 per lb., as the life of the tire is dependent on the tread wear.

In terms of a vehicle the average tire cost is 1.488 cents per mile on a crushed rock or gravel road, and 0.388 cents on a cement road. On this basis, as shown in Table 6, the yearly saving in favor of the concrete road is from $2 000 for a traffic of 500 vehicles to $6 025 in the case of the traffic reaching 1 500 vehicles per day. The unit fuel cost adopted was 1.43 cents per mile for a gravel or crushed rock road and 1.09 cents per mile for concrete.

Extending these computations gave some rather amazing figures. For the concrete road the accumulated saving in tire wear at the end of 20 years amounts to $136 456, while the corresponding saving in gasoline is $42 090.

Making a recapitulation of these items, and paying no attention to savings due to time losses, wear and tear on the vehicle, etc., gives the following:

In Favor of Concrete:	
Maintenance	$14 788
Tires	136 456
Gasoline	42 090
Total	$193 334
In Favor of Gravel:	
Interest on construction	$21 382

With the assumptions and the interest charges noted, the difference shows a saving for concrete of $171 952 per mile in a 20-year period.

If $4 935 is deposited in a bank every year, drawing interest at 5½%, compounded annually, it will amount to $172 000 at the end of 20 years. In other words, under the traffic conditions assumed a concrete road plus $4 935 saved annually is equivalent to the cost of a gravel road. These figures are so staggering that they appear quite fallacious. This is because of the habit of neglecting to recognize the indirect taxes paid for highways. Bills for tires, gasoline, and repairs, are part of the operating costs.

There is a grave question whether it is proper to charge interest on savings due to maintenance and operating costs. Omitting this item, but leaving it intact for interest on the original investment, the total saving due only to maintenance, tires, and gasoline, would amount to $120 617 less $21 382 due to the original investment, leaving a net saving in favor of the concrete of $99 235 per mile (Table 6). The annuity in this case then would be $2 850 instead of $4 935. In other words, without the interest, concrete road could be constructed and the sum of $2 850 a year set aside annually from the savings so that at the end of a 20-year period there would be $99 000 in the bank for future improvement on the mile of road in question.

TABLE 6.—TIRE AND GASOLINE COSTS PER MILE OF ROAD FOR 500 TO 1 500 VEHICLES PER DAY.

Year.	Estimated traffic per day.	Gravel Roads.		Concrete Roads.		Saving for Concrete Road.					
						Tires.			Gasoline.		
		Tires.	Gasoline.	Tires.	Gasoline.	Yearly saving.	Interest at 5½ per cent.	Accumulative saving.	Yearly saving.	Interest at 5½ per cent.	Accumulative saving.
1	500	$ 2 710	$ 2 610	$ 708	$ 1 990	$2 002	......	$2 002	$ 620	......	$ 620
2	570	3 090	2 970	808	2 268	2 282	$ 110	4 394	702	$ 34	1 356
3	630	3 425	3 290	893	2 510	2 532	241	7 167	780	73	2 209
4	700	3 800	3 650	992	2 786	2 808	394	10 369	864	121	3 194
5	770	4 180	4 020	1 092	3 065	3 088	570	14 027	955	175	4 324
6	835	4 540	4 360	1 184	3 325	3 356	771	18 154	1 035	238	5 597
7	890	4 830	4 650	1 261	3 540	3 569	1 000	22 723	1 110	307	7 014
8	960	5 210	5 010	1 360	3 821	3 850	1 250	27 823	1 189	386	8 589
9	1 020	5 540	5 320	1 446	4 060	4 094	1 530	33 447	1 260	472	10 321
10	1 080	5 870	5 630	1 530	4 300	4 340	1 830	39 617	1 330	568	12 219
11	1 140	6 190	5 950	1 615	4 540	4 575	2 180	46 372	1 410	672	14 301
12	1 195	6 490	6 230	1 694	4 760	4 796	2 550	53 718	1 470	787	16 558
13	1 250	6 790	6 520	1 770	4 975	5 020	2 950	61 688	1 545	910	19 013
14	1 300	7 060	6 780	1 843	5 170	5 217	3 390	70 295	1 610	1 045	21 668
15	1 350	7 340	7 050	1 912	5 370	5 428	3 860	79 583	1 680	1 190	24 538
16	1 385	7 530	7 230	1 962	5 510	5 568	4 370	89 521	1 720	1 350	27 608
17	1 430	7 770	7 460	2 026	5 690	5 744	4 920	100 185	1 770	1 522	30 900
18	1 460	7 930	7 620	2 068	5 810	5 862	5 500	111 547	1 810	1 700	34 410
19	1 480	8 050	7 720	2 096	5 890	5 954	6 130	123 631	1 830	1 890	38 130
20	1 500	8 150	7 830	2 125	5 970	6 025	6 800	136 456	1 860	2 100	42 090
		$116 495	$111 900	$30 385	$85 350	$86 110	......	......	$25 550	......	......

RECAPITULATION:

Saving Over 20-Year Period:		No Interest Computed:	
Maintenance	$14 788.00	Saving in maintenance	$ 8 957.00
Tires	136 456.00	" " tires	86 110.00
Gas	42 090.00	" " gas	25 550.00
Total	$193 334.00	Total	$120 617.00
Less loss due to difference in interest on construction	21 382.00	Less	21 382.00
Total saving	$171 952.00	Total savings	$99 235.00
Annuity	$4 935.00	Annuity	$ 2 850.00

TABLE 7.—MAINTENANCE AND CONSTRUCTION COSTS ON 300 TO 1 000 VEHICLES PER DAY.

Year.	Traffic, in cars per day.	Annual traffic, in thousands of tons.	Comparative Costs of Maintenance. Gravel: $M = 400 + \frac{A\,T}{3\,000}$.	Comparative Costs of Maintenance. Concrete: $M = 100 + \frac{A\,T}{6\,000}$.	Comparative Costs of Maintenance. Difference in maintenance cost.	Interest at 5½ per cent.	Accumulated savings.	Construction Costs. Gravel. Initial and reconstruction costs.	Construction Costs. Gravel. Interest at 4½ per cent.	Construction Costs. Gravel. Total accumulated costs.	Construction Costs. Concrete. Interest at 4½ per cent.	Construction Costs. Concrete. Accumulated costs.
1	300	164	$455	$127	$328	......	$ 328	$4 000	......	$ 4 000	.....	$24 000*
2	350	192	464	132	332	$18	678	......	$180	4 180	$1 080	25 080
3	395	216	472	136	336	37	1 051	......	188	4 368	1 130	26 210
4	440	241	480	140	340	58	1 449	......	197	4 565	1 180	27 390
5	486	263	488	144	344	80	1 873	1 500	206	6 271	1 230	28 620
6	530	290	493	149	344	108	2 320	......	282	6 553	1 290	29 910
7	575	315	505	152	353	127	2 800	......	295	6 848	1 340	31 250
8	620	340	513	157	356	154	3 310	......	308	7 156	1 410	32 660
9	665	364	521	161	360	182	3 852	......	322	7 478	1 470	34 130
10	710	389	529	165	364	212	4 428	1 500	336	9 314	1 540	35 670
11	740	405	535	167	368	243	5 039	......	420	9 734	1 600	37 270
12	780	427	542	171	371	277	5 687	......	438	10 172	1 680	38 950
13	825	452	551	175	376	312	6 375	......	457	10 629	1 750	40 700
14	860	471	557	178	379	350	7 104	......	477	11 106	1 830	42 530
15	890	487	562	181	381	390	7 875	1 500	500	13 106	1 910	44 440
16	925	507	569	184	385	433	8 693	......	590	13 696	2 000	46 440
17	945	517	572	186	386	477	9 556	......	616	14 312	2 090	48 530
18	970	531	577	188	389	525	10 470	......	644	14 956	2 190	50 720
19	985	540	580	190	390	575	11 435	......	673	15 629	2 280	53 000
20	1 000	547	582	191	391	630	12 456	......	703	16 332	2 380	55 380
								Salvage value..	......	1 500	......	15 000
....	....	....	....	....	$7 273	......	......	......	......	$14 832	....	$40 380
....	....	....	....	....	....	......	......	......	......	....	....	14 832
Difference........	....	....	....	....	....	......	......	......	......	......	......	$25 548

* Initial cost.

TABLE 8.—TIRE AND GASOLINE COSTS ON 300 TO 1 000 VEHICLES PER DAY.

Year.	Traffic, in cars per day.	Gravel Roads.		Concrete Roads.		Saving for Concrete Road.					
						Tires.			Gasoline.		
		Tires.	Gasoline.	Tires.	Gasoline.	Yearly saving.	Interest at 5½ per cent.	Accumulative saving	Yearly saving.	Interest at 5½ per cent.	Total accumulative saving.
1	300	$1 628	$1 560	$425	$1 194	$1 203	......	$1 203	$366	......	$366
2	350	1 900	1 830	496	1 390	1 404	$65	2 672	440	$20	826
3	395	2 140	2 060	560	1 570	1 580	147	4 399	490	45	1 361
4	440	2 390	2 300	624	1 750	1 766	241	6 406	550	75	1 986
5	480	2 610	2 500	680	1 910	1 930	352	8 688	590	109	2 685
6	530	2 870	2 770	751	2 110	2 119	477	11 284	660	147	3 492
7	575	3 120	3 000	815	2 290	2 305	620	14 209	710	192	4 394
8	620	3 360	3 240	880	2 470	2 480	782	17 471	770	241	5 405
9	665	3 610	3 470	943	2 650	2 667	960	21 098	820	297	6 522
10	710	3 850	3 700	1 006	2 820	2 844	1 158	25 100	880	359	7 761
11	740	4 020	3 860	1 048	2 940	2 972	1 380	29 452	920	427	9 108
12	780	4 230	4 070	1 105	3 100	3 125	1 620	34 197	970	500	10 578
13	825	4 470	4 300	1 168	3 280	3 302	1 880	39 379	1 020	580	12 178
14	860	4 660	4 490	1 217	3 420	3 443	2 160	44 982	1 070	670	13 918
15	890	4 830	4 650	1 260	3 540	3 570	2 470	51 022	1 110	765	15 793
16	925	5 020	4 830	1 310	3 680	3 710	2 810	57 542	1 150	869	17 812
17	945	5 130	4 930	1 340	3 760	3 790	3 160	64 492	1 170	980	19 962
18	970	5 260	5 060	1 374	3 860	3 886	3 550	71 928	1 200	1 098	22 260
19	985	5 350	5 140	1 395	3 920	3 955	3 950	79 833	1 220	1 224	24 704
20	1 000	5 430	5 220	1 417	3 980	4 013	4 390	88 236	1 240	1 360	27 304
Total......	......	$75 878	$72 980	$19 814	$55 634	$56 064	......	......	$17 346	......	......

RECAPITULATION:

Saving Over 20-Year Period:		Without Interest:	
Maintenance	$12 456.00	Saving in maintenance	$7 273.00
Tires	88 236.00	" " tires	56 064.00
Gas	27 304.00	" " gas	17 346.00
Total	$127 996.00	Total	$80 683.00
Less	25 548.00	Less	25 548.00
Total savings	$102 448.00	Total savings	$55 135.00
Annuity	$2 930.00	Annuity	$1 581.00

As a matter of comparison other figures were computed based on a traffic range from 300 to 1 000 vehicles per day, using the same tire and gasoline costs per mile (Tables 7 and 8). With this traffic density the saving on the concrete road, if interest were computed, would be $102 000, or the equivalent of a yearly annuity of $2 930. If the interest was omitted from the operating costs the net saving in 20 years would be $55 000, or an annual annuity equivalent to $1 580.

It can be readily seen that the original cost of the surface means very little in comparison with the operating cost. The writer realizes that these operating costs are difficult to compute. Nevertheless, no matter what the basis, anything that tends to reduce the operating costs will rapidly run into large totals. It would be of interest to make similar computations relative to other types than the two that have been here treated.

The writer wishes to acknowledge the assistance of Mr. Frank Cnare, Plan Engineer of the Wisconsin Highway Commission, in making the computations.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

HORIZONTAL CONTROL IN SURVEYING

PRELIMINARY REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION ON HORIZONTAL CONTROL*

By George L. Hosmer,† M. Am. Soc. C. E.

The term, "Horizontal Control", as applied to surveying is used to designate any system of triangulation or traverse (usually established in advance of the detailed work) which serves to insure the standard of accuracy of the entire survey. Whenever a survey is to be extended in any direction, or details are to be filled in, it is usual to start a traverse from one of the accurately located stations of previously executed control and to close on another. Since these stations can be located with almost any desired degree of accuracy, and since the error of closure indicates the accuracy of the traverse itself, the general accuracy of the survey may thus be kept from falling below any specified limit. Since each new traverse starts from an accurately located point, the errors of other traverses have no effect on it.

From the standpoint of accuracy such a system of control is most satisfactory, but at first it usually appears elaborate, difficult, and expensive. Hence, it is important that the engineer should appreciate the true relation of control to the entire survey.

The requirements of the survey itself appear to be simple, merely permitting an error not greater than 1 part in 10 000, for instance; or, so that the position of a point will not be in error by more than, perhaps, 0.10 ft. Therefore, one is tempted to ask: "Is this triangulation an unnecessary refinement just to show what can be done with fine instruments?"

In order that a survey may fulfill its purpose the measurements must be made with an accuracy sufficient for the end in view, that is, the "errors of

Note.—Written discussion on this report will be closed in March, 1929, *Proceedings*.

* Presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928. Approved by the Executive Committee of the Surveying and Mapping Division and by the Division Committee on Control Surveys, Messrs. C. V. Hodgson, *Chairman*, E. M. Douglas, George L. Hosmer, A. B. Pierce, and R. H. Randall.

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closure" must not exceed a certain amount. Errors of measurement always accumulate to some extent; they seldom balance exactly. The accuracy of locating the controlling points should be such that, like a structure, the control will be strong enough to carry all parts of the subsequent survey, and nowhere become weaker than the amount specified as the limit. The control thus holds all the parts rigidly in place. The question, therefore, may be answered by asking another: "Is an engineer justified in putting in less than the amount of steel demanded for safety in order to reduce the total cost of a structure, or to enable him to put more money into its ornamentation?"

Two reasons should make it clear that control is a practical necessity. First, the usual procedure—beginning a survey at the point where information is most urgently needed and then adding to it, bit by bit, and yet striving to keep the accuracy of the whole up to the specified standard—has proved to be almost impossible. Even where accomplished, it is expensive because one must either repeat a great deal of work or else do all the work much more accurately. The control itself, as distinguished from the subsequent survey, is expensive only in the beginning. As the work proceeds, the relative cost diminishes, because the same control serves a greater area, and soon, by saving in the detail work, it pays for itself, often many times over.

Second, without adequate control, the difficulties and delays occurring because of poor checks and the necessity for deciding at every point which of several values to adopt, are serious. Values have to be revised continually, due not only to inaccuracies, but even more to inconsistencies. After converging on some line by two different routes, the computer is sure to find discrepancies because at the start the base lines were inconsistent. He is certain to be faced, sooner or later, with the situation that two lines, comparatively near each other, have computed lengths that are relatively in error by a much larger amount than would be obtained by running directly from one to the other. This makes the survey appear much less accurate than it really is.

The measurements of the control are made with high accuracy, but they are necessarily subject to small errors. If this system is adjusted by the method of least squares, the first result is that the geometric relations existing among angles and sides, not quite exact in the measured angles, are made perfect. The length of a line as derived by calculation from some base will then agree exactly with that length derived from any other base, a result which could hardly ever happen if no adjustment were made. The second result is that of all the possible ways in which this agreement might be brought about that particular one is used which gives the best (most probable) values for the angles and distances. This adjustment does away with endless difficulties in trying to harmonize results by inadequate methods.

Thus, basing a survey on a well-planned control is not a matter of adding frills for the sake of appearances, but of planning the whole survey so as to obtain strength and economy. The control surveys may be planned so that the adjustment will be fairly simple and inexpensive. Once done properly, it is done for all time, and the engineer knows he can start from any line and close on any other line with accuracy.

CONTROL SURVEYS—GRADES OF TRIANGULATION AND TRAVERSES

Systems of triangulation or of traverses for controlling the accuracy of surveys are divided into four grades, designated as first, second, third, and fourth order. The basis of classification is the accuracy finally reached.

Extensive surveys involve the use of triangles of different sizes and for different purposes, the larger constituting the main control and the smaller governing the detailed work, such as topographic surveys. Formerly, the terms, "primary", "secondary", etc., were used to designate the classes of triangulation established for different purposes, as well as to distinguish different degrees of accuracy. This resulted in some confusion. It is now customary to speak of the "main scheme" of triangulation (sometimes the "principal" triangulation) and the "subsidiary" triangulation. In some kinds of work the main scheme, therefore, may be of the second or of the third order of accuracy.

First-order triangulation is that in which the average error of closure of a triangle is about 1″, or less, and the maximum error, seldom greater than 3″. As to length, the error of closure between bases must not exceed 1 part in 25 000, after the adjustment for angle and side equations has been made.

First-order traverse is that in which the discrepancy in the position of a junction station, where a loop is closed by connecting with first-order adjusted triangulation or traverse, does not exceed 1 part in 25 000.

In second-order triangulation, the limits are 3″ and 6″ in angle, and 1 part in 10 000 in length. In second-order traverse, the position check should not show an error greater than 1 part in 10 000.

The corresponding quantities for third-order triangulation are 5″, and 10″; and 1 part in 5 000. Third-order traverse must have a position check with an error not greater than 1 part in 5 000.

To the present time fourth-order triangulation and traverse have included all work that fell below the limits for third-order accuracy, but no specifications have ever been written for this class. It is felt by the Committee on Control Surveys that such specifications should be written, and the Committee expects to undertake this work.

In the preceding classification, the "error of closure" is understood to mean the amount by which the sum of the three measured angles differs from the theoretical sum, that is, 180° plus the spherical excess of the triangle. The "closure in length", or "base check", means the actual difference between the length of a base as computed from the preceding base, and as measured or as given by a previous adjustment, divided by the length of the base itself, it being understood that the angle and side equations of the net have been satisfied, but that the adjustments for base, azimuth, and position have not been made. The "position check" is expressed by dividing the distance between the position from a new traverse and the old (standard) position by the distance run in the new traverse.

This classification was adopted in 1925 by the Federal Board of Surveys and Maps. Previous to 1921, triangulation had been classed as primary, secondary, and tertiary—simply relative terms. During the period between

1921 and 1925, the expressions, precise, primary, secondary, and tertiary, were in use.

TRIANGULATION

A triangulation system consists of a network of triangles, the length of one side of one triangle being known. A sufficient number of angles are measured to make possible the calculation of all the other sides in the net. To furnish adequate checks, other angles, distances, and azimuths should be measured. For example, all three angles of each triangle should be measured; the geometric figures used should permit of calculating the length of a line in more than one way; additional base lines should be introduced at intervals; and astronomical azimuths should be observed as often as required to prevent undue accumulation of errors in the calculated azimuths. Such a system ordinarily furnishes the most accurate and economical means of carrying distances from one region to another, and of establishing control over an extended area.

Geometric Figures of a Triangulation.—The three commonest geometric units in a triangulation net are (1) the simple triangle; (2) the quadrilateral; and (3) the polygon with an interior station. The chain of simple triangles will span a distance with the least expenditure of time in measuring angles; it is not, however, a strong figure, since there are relatively few checks on the accuracy. A pentagon, or hexagon, with an interior station is a strong figure and also covers a comparatively large area. Quadrilaterals consisting of four stations joined by six lines, and having eight measured angles, are still stronger and are generally preferred for a belt of triangulation.

Strength of Figures.—The strength of triangulation, as measured by the "probable error" of the (computed) logarithm of a triangle side, depends on (1) the relative number of measured directions and geometric conditions; (2) the magnitudes of the angles of the individual triangles; and (3) the precision of the measurement of the directions or of the angles. The relative strength of two alternative belts of triangulation will depend on the first and second of these three factors, the third being the same for both. The formula for computing what is called the "strength factor", R, is as follows:

$$R = \frac{D - C}{D} \left(\delta_A{}^2 + \delta_A\,\delta_B + \delta_B{}^2\right) \ldots\ldots\ldots\ldots\ldots (1)$$

in which,

D = the number of directions to be determined.
C = the number of geometric conditions.
δ_A = the difference for 1″ in the log sin of the angle, A.
δ_B = the difference for 1″ in the log sin of the angle, B.

When there are several values for R for the same net they may be designated as R_1, R_2, etc., R_1 being the smallest. Two schemes of triangulation may be compared as to strength by comparing their values, ΣR_1, that having the smaller ΣR_1 being the stronger chain. In case R_1 is nearly the same for both, then the values of R_2 may be compared.*

* For tables and examples, see U. S. Coast and Geodetic Survey *Special Publication* No. 120.

Frequency of Bases.—The accuracy of the triangulation in length may be controlled by specifying that the values of ΣR shall not exceed a certain amount before a new base is introduced. In first-order triangulation, the value of ΣR_1 is about 80. This will correspond to from ten to twenty triangles, according to the strength of the individual figures secured. If there is difficulty in securing a base site, ΣR_1 may approach but not exceed 110. On second-order triangulation the desirable and limiting values for ΣR_1 between bases should be from 100 to 130, respectively, while on third-order triangulation the values should be from 125 to 175.

Equation (1) applies strictly to a belt of triangulation, and shows conditions which will give a length with accuracy. When the triangulation covers an area and there are consequently many closures of loops, this method of testing the accuracy becomes more complicated. The calculations may be simplified, however, by specifying that the sum of the values of R_1 from the nearest base to any line of the triangulation through the strongest chain of simple figures shall not exceed a certain quantity.

For a detached system of triangulation of any considerable extent, it is best to have a base at the beginning and the end of the net, rather than to depend on a single base at the center, otherwise an error in the measurement of the base might remain undetected.

Accuracy of Base Lines.—The measurement of a base line with modern methods is a comparatively simple operation. In a first-order base the total actual error* should be less than 1 part in 300 000 of the length of the base, and the "probable error" of the total length should be less than 1 part in 1 000 000. For a second-order base the corresponding errors have been considered to be less than 1 part in 100 000 and 1 part in 300 000, respectively; and for a third-order base, 1 part in 30 000 and 1 part in 100 000.

Selection of Triangulation Stations.—In selecting triangulation stations, careful consideration should be given to:

1.—The distribution of the stations to cover the area so that the final results may be of the greatest utility.

2.—The accuracy of the results, as affected by the type of figure chosen and the shape of the individual triangles involved.

3.—The location of base lines and a strong connection with the triangulation.

4.—The cost of executing the control.

Distribution of Stations.—The number of stations to be established and the distances between them depend on the nature of the survey. At the time when the triangulation or traverse is being established additional survey points may be put in at little extra cost. In so far as it is possible to foresee the needs, it is desirable to locate such stations at this time, even if it slows down the control work. These stations may often be located by intersecting on points which are not to be occupied with the theodolite, from at least three main stations. If subsidiary stations have to be put in later, it means extra expense to re-occupy several principal stations to measure the angles. These

* This is obtained by estimating first the error which may enter from each possible source and then combining these by the usual least square formula for resultant errors.

points are used so frequently later that the first cost of their location is relatively insignificant.

Where towers have to be used during the observing and no well defined object or other located station can be seen from the ground at a station, an azimuth mark should be established at a distance of at least several hundred feet in order that an azimuth may be available on subsequent local surveys.

Traverse

Under some conditions triangulation is not the most economical method of establishing control. In flat and heavily timbered country, the triangulation is at a disadvantage: First, because of the expense of building high towers; and, second, because the short length of line necessitates a large number of figures to cover the required distance or area. Although the ordinary traverse is not of a high order of accuracy, traverse may be conducted so as to give nearly the same final accuracy as triangulation.

When practicable, the traverse should be run along a railway line, so as to obtain fairly long sights with a minimum of brush cutting. Stations (properly marked) should be left at railroad crossings and at towns and villages for local use.

With short lines, the azimuth errors accumulate unduly. When a station is near a town, the next (adjacent) station should not be more than, say, two miles away, so that an azimuth may be obtained conveniently for local surveys. Where these two conditions conflict, the azimuth may be carried through on long lines, but with short lines left for local use.

On traverses of the highest order of accuracy, three different invar base-line tapes are used, one being kept for a field standard. Distances should be checked against large mistakes with a 300-ft. steel tape.

As a general rule the tape is laid on top of a rail. When passing from the rail to a station not on the railway line, such as a vertex between tangents, the measurement should be made on regular base-line stakes. Measurements on rails in very wet weather are found to be inaccurate and should be avoided. Angles should be measured with the best theodolite available, and with the same care as for triangulation of the same order.

Owing to the relatively large number of lines in a traverse as compared with triangulation, it is necessary to observe on Polaris frequently for azimuth, since the errors of calculated azimuths tend to accumulate rapidly. In southern latitudes where Polaris is not visible, other stars must be used.

On first-order traverse an azimuth should be observed every 10 to 15 stations, and the probable error of the result should be about 0.50″ (for "Laplace" stations,* the probable error should not exceed 0.30″); on second-order traverse the frequency should be every 15 to 20 stations, and the probable error about 1.5″; and, on third-order traverse, every 30 to 50 stations, and the error, 5″.

When possible, the geodetic azimuth should be carried through the traverse by rather long sights, if necessary using a separate series of stations, while the measurement of distance may be conducted on shorter lines.

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* See p. 2499.

Triangulation and Traverse Compared

Under ordinary circumstances a distance may be spanned by triangulation with the greatest accuracy and the least work. It is more independent of the character of the country. Traverses, on the other hand, must be carried across the intervening country, so that lakes, rivers, swamps, and dense woods present practical obstacles which may be insurmountable. Ordinarily, more work is required to obtain the same accuracy from traverses as from triangulation, especially if the distance covered is large. Traverses will also require more azimuth control.

When the area is flat and wooded, triangulation loses its advantage. The lines must be shorter, and every station will require a high, expensive, observing tower. In such country, if the traverse can be run along a railway, or even a highway, it is likely to prove more economical than triangulation, and have the advantages of leaving for local use many stations comparatively close together.

Accuracy Required

In deciding on the accuracy required for the control, it is important to consider that the demand in the future may be greater than at present. If the survey is for a topographical map the decision may be easy. In a city survey the control is intended to be permanent, and since there is a tendency for land values to increase, too low an accuracy in the beginning may later necessitate starting all over again.

There is perhaps a general impression that the triangulation used to control the general map of the country is of a high enough accuracy for any local needs. The principal function of the National triangulation is to furnish accurate geographic positions, and the accuracy of individual lines is not necessarily sufficient where property values are very high. In such circumstances one or more local bases must be measured.

Monumentation

To an engineer, it is a convenience at all times to have reliable monuments from which to work, but the great majority of surveys are not adequately monumented. As a result great and needless expense is often entailed in making re-surveys. The monumentation should be carried out at the beginning of the survey and not left until later. This may not be so fully appreciated, however, by the non-technical man, on account of the expense involved, and he may require it to be postponed or given up altogether. To understand its importance, the cost of setting monuments has only to be balanced against the expensive re-locating of lost points from distant stations correctly monumented.

On isolated surveys for engineering projects where the monumentation is all the more imperative because there are no other marks from which to work, it is most often neglected on account of the first cost, and perhaps by failure to look ahead to the time when it may be of great importance. All principal triangulation and traverse stations should be marked in a permanent manner, similar to Government surveys.

Geodetic Datum

Like elevations, geodetic positions located by triangulation or traverse must refer to a specified surface and the station must be assigned definite positions on that surface. Having selected some station and assigned to it a latitude and a longitude, and having assumed the azimuth to some adjacent triangulation station, the positions of all other stations may be derived by calculation.

The datum for geodetic positions in the United States is that formerly (1901) known as the "United States Standard Datum" and since 1913 as the "North American Datum". The triangulation station, "Meade's Ranch" in Kansas, is the initial point.

Since the Bessel spheroid of 1841 to which earlier Government control surveys were referred, did not fit accurately the surface of the United States, a change was made about 1880 to the "Clarke Spheroid of 1866". As surveys were still further extended, it became necessary to decide on the best position for the triangulation on the Clarke spheroid. Latitude and longitude were finally chosen for the initial point so as to give a close fit to its true position as shown by the latitudes and longitudes observed astronomically.

When taking published Government geodetic positions for beginning a survey, it is important to ascertain whether they are on the Standard Datum described. For example, positions published before 1880 should not be used until they have been verified. Those given in special investigations, such as the Transcontinental Arc along the Thirty-ninth Parallel, are not on the Standard Datum. The safest procedure is to obtain the most recently published list from the Director of the U. S. Coast and Geodetic Survey.

Datum for Surveys of Small Areas.—Any survey in which points are located by geographical co-ordinates (latitude and longitude) should be referred to the Standard Datum. In city surveys the rectangular co-ordinates are often defined with reference to some triangulation point or some conspicuous monument, at which the fundamental plane is assumed tangent to the Clarke spheroid.

The base lines may be derived from triangulation, but can often be more accurately obtained by direct measurement. Azimuths may best be determined by direct observation, especially if the triangulation has been carried a long distance from an azimuth station; but even if the lengths and directions are measured directly, it is still important to obtain the position of the survey as a whole by triangulation stations referred to the Standard Datum. As surveys are extended, and ultimately joined, this matter becomes increasingly important.

In order to avoid negative values, the fundamental (tangent) point may be assigned, in the unit of length used, such co-ordinates as 100 000 N, and 100 000 E. The "grid" system in use in military surveys and other operations is a plane system similar to the foregoing.*

* For convenient methods of converting geographical co-ordinates into plane co-ordinates, reference is made to *Special Publication No. 71,* U. S. Coast and Geodetic Survey, entitled "Relation Between Plane Rectangular Co-ordinates and Geographic Positions."

Azimuth Orientation

The computed azimuths of triangulation lines, as found by calculation, using the measured angles of the triangles, are subject to relatively large errors. These errors that affect the azimuths are not wholly of an accidental character but tend to twist the triangulation, usually in the same direction. This is due in part to the fact that in day-time observations the theodolite is likely to be heated more on one side than on the other, which introduces systematic errors into the readings of the graduated circle. In order to prevent an accumulation of these errors it is necessary to strengthen the triangulation by introducing additional astronomical azimuths at frequent intervals.

Since all observed azimuths are subject to the effect of any local deflection of the vertical which may exist, they will not be of the highest order of accuracy unless they can be freed from the effects of this error. At stations of first-order triangulation and traverse where the longitude can also be observed, it is easy to determine the amount of this deflection and apply a correction to the observed azimuth, thus obtaining a correction to be applied to the computed azimuth. The observed azimuth as corrected is as accurate as any other azimuth in the survey. Stations at which both azimuth and longitude are observed are called "Laplace stations", and the azimuths corrected in this manner are known as "Laplace azimuths". With the radio receiving apparatus now in use, it is possible to observe longitude at any point to which the instruments can be transported. Triangulation, therefore, can be kept accurately oriented without great expense. It has been found advisable to introduce such azimuths every six or eight figures in a belt of triangulation of the first order. The accuracy required in azimuth observations for the different grades of work has been given, and the same limits of error apply to the corresponding classes of triangulation.

Conclusion

To sum up the discussion, the adequate control of an area requires that:

(1) Triangulation or traverse stations shall be so distributed that from any point within the area it will not be necessary in detailed surveys to go more than a certain specified distance to reach a control station. These stations should be, if possible, in positions which will afford a maximum of usefulness.

(2) The accuracy of the lengths and azimuths of the lines, and the positions of the points, shall be reliable within the limits specified for the particular kind of survey in question.

(3) The points should be permanently marked and clearly described.

(4) Stations should be near enough together so that it is possible to start local surveys by means of a distance and an azimuth as well as a position.

(5) The survey should be referred to the North American Datum.

In order to accomplish these results, it is necessary that the reconnaissance be thoroughly made by one who is competent to do this work, and who appreciates the requirements of the particular kind of survey in question.

The measurements should be made so that each operation is carried out with the care necessary to give the accuracy specified for the final result. The greatest accuracy is to be placed where it is most needed, and cost saved where it is really permissible without loss of accuracy.

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PAPERS AND DISCUSSIONS

PRECISE WEIR MEASUREMENTS

Discussion*

By Clemens Herschel, Past-President, Am. Soc. C. E.

Clemens Herschel,† Past-President, Am. Soc. C. E. (by letter).‡—"Daniel Bernouilli often said to me to eschew all complicated formulas; he believing that the organization of Nature is too simple to lead to them; and should one find such, the explanation is that one's computations were based upon false hypotheses."§

"These things are beyond all use,
And I do fear them."‖

In reading the paper, and the learned discussions upon it, the question is likely to arise in the mind of a thoughtful hydraulic engineer:

"What is to be the outcome of so much labor, toward serving the profession in its work? Is there any use in it all? What is or should be a weir? Should it be the tool of an engineer, or is it a laboratory instrument, on which to found experiments and computations, and diagrams, and the laborious setting up of formulas to encompass these experiments? Should not an engineer, of all professions, constantly keep utility in view? And, surely, is not simplicity a measure of utility?"

Of course, every one knows the argument that science can not be fettered by considerations of utility; that science must be pursued for science's sake, and motives of utility must be rigorously banished; leading, as a possibly fine example, to the work of the German professor who spent a life time in translating the Latin classics into ancient Greek. But are such considerations properly presentable to engineers? Are not in these days, as many and more inventions and discoveries made by those who directly and specifically seek them, as by those who blindly grope and experiment, even though they do happen occasionally to lay the foundation for the utilitarian work of others?

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* Discussion of the paper by Ernest W. Schoder, M. Am. Soc. C. E., and the late Kenneth B. Turner, Esq., continued from August, 1928, *Proceedings*.

† Hydr. Engr., New York, N. Y.

‡ Received by the Secretary, August 13, 1928.

§ R. Wolf, "Biographies Relating to the History of Civilization in Switzerland", 1860, Vol. III, p. 176 (in French).

‖ Shakespeare, "Julius Caesar", Act II, Scene 2.

Consider then, that a notable part of the work of a hydraulic engineer is to measure the flow of water, as he meets with it in his practice. This will include the flow of water in pipes, and in natural channels of a much larger size, such as rivers; and but little else. For the case of pipes a meter has been in existence since 1887, applicable alike to pipes ¼ in., or an unlimited number of feet (17 ft. actually in use) in diameter, the discharge formula of which is expressed in the simplest of terms, and is usually made to record its results automatically. The writer has lately seen a ponderous diagram proving once again that its record is correct, and in this particular case, that it coincides with the results found by the "salt velocity method"; or, as it might be put, "the salt velocity method" has been found accurate by comparison with the Standard Venturi meter.

The meter need not be permanently installed. A temporarily installed Venturi meter, built of wood, except the throat-piece, will in most cases cost less than the long drawn out operations made necessary by following almost any other method of work. Of course, there are exceptions, although not many, generally to be covered by the Gibson, or by the salt velocity method; but as a rule the sharp-crested weir has long ago become needlessly laborious in itself; needless, and useless, even obsolete, as an instrument to meter flowing water.

Not long ago the writer received from the Director of the Delta Barrage, in Egypt, a little book entitled: "The Discharge of Pipes", which described experiments made to determine the discharge of forms of pipe used in Egypt to measure irrigation water. It was founded, no doubt, on Egyptian habits, apparently in vogue since the time of the Pharaohs; but this little book, to the mind of the writer, is a good parallel to present-day dissertations on weirs and weir formulas. Their use seems to him to be as much out of date for metering water, as that of the forms of pipe used presumably for thousands of years past in Egypt, for the same purpose.

This is the age of machinery and mechanisms, as compared with ages of manual and mental work in the past. Offices are already filled with computing machines, typewriters, dictaphones, cameras, and what not besides; similarly much mechanical work of the hydraulic engineer in the field can likewise be replaced by the work of meters and their recording mechanisms. Take the case of the operation of the Aquedotto Pugliese, in Italy, which in its work of keeping what was nigh a desert, a flourishing district, uses 400 Venturi meters; or take New York's main water supply, that has three 17-ft. meters and more than 70 of smaller size.

To resume the consideration of weir formulas: Let any one look at the collection of such in the paper:

$$\mu_0 = 0.605 + \frac{1}{1\,020\,h_0 - 3} + 0.08\,\frac{h_0}{d_0}$$

for the value of one term to be used in computing according to the formula that contains it; and ask himself the question: Whether this is not the case of "a poor sport that is not worth the candle".

The defects of all weir gaugings are not new matters for the consideration of hydraulic engineers; and it is both startling and discouraging to find how the present paper and discussion have completely ignored what was said on the subject by Floyd A. Nagler, M. Am. Soc. C. E., in 1918* and by the writer in 1920,† and the reader will wonder at the dilatoriness with which progress is made in the practical advance of the art of the hydraulic engineer.

As regards the paper† last referred to, it must be remembered that the experiments were made in an hydraulic laboratory, whose use, including installation of apparatus and its removal, of necessity counted only 23 days. They consisted of devising a more rational form of weir than the classic sharp-edged weir; together with an attempt to measure "head on the weir" with "correction for velocity of approach" automatically compensated for; and resulted in finding a weir the formula of discharge of which was:

$$Q = 5.50d \text{ (English measures)}$$
$$Q = 1.675d \text{ (metric measures)}$$

The exactitude of this result has been questioned by a group of students in the University of California; but merely to have made an approach to such a result, is already a great advance in the art. Naturally, the apparatus used, the first of its kind, is capable of palpable improvements; and should meet with them at the hands of the next experimenters on the "hollow crest weir". This has been since May, 1920, repeatedly expressed to inquiring friends, and for future possible service an extract from one such letter follows:

"I have learned of the very great importance of having the piezometer holes in the crest tube bored precisely at a right, or a proposed angle to the '2:1 slope'.‡ And I regret to state that this direction of bore was not done with a mechanical direction of the boring instrument, but made only with direction guided by hand and eye, in case of the weir I built in Boston. There were also too many of these piezometer holes, so that one cannot now tell which hole or holes prevailed, and the precise angle at which it or they were bored.

"I believe, were I in position to repeat the experiments, I would follow the 'original intention'* and have only one piezometer hole—certainly not more than three. This would surely lead to a hollow crest weir, having a straight line formula. Whether it would also prove the same, to have the piezometer hole, or holes, at right angles to the 2:1 slope, I cannot now affirm.

"Another point: Referring to Figs. 4 and 5,§ you will notice the lack of any and all test, in the true sense of the word, of an idea; the idea being, to correct mechanically—by the way the observations are taken—for 'velocity of approach'. I still think this apparatus is good, and that the idea has value."

To summarize:

(1) As the late Frederic P. Stearns, Past-President, Am. Soc. C. E., expressed it,‖ "in the present state of the art the weir is not an accurate instrument for the measurement of water"; and no improvement in its use has been made, since that was written.

* "Verification of the Bazin Weir Formula by Hydro-Chemical Gaugings," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 105.

† "An Improved Form of Weir for Gauging in Open Channels," *Transactions*, Am. Soc. Mech. Engrs., Vol. 42 (1920), p. 191; see, also, *Engineering News-Record*, June 24, 1920.

‡ *Transactions*, Am. Soc. Mech. Engrs., Vol. 42 (1920), No. 36, p. 205.

§ *Loc. cit.*, pp. 208–209; see, also, Nos. 42–45, pp. 207–208.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 172.

(2) The weir, and especially the sharp-crested weir, is, or should be, grossly obsolete so far as cost, convenience, and time-saving in its use to meter water are considered.

(3) Experiments upon its discharge, and setting up of involved formulas to portray with more or less precision their results will not remove these defects; nor are they of use to the practicing hydraulic engineer.

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PAPERS AND DISCUSSIONS

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THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

Discussion*

BY D. P. KRYNINE, M. AM. SOC. C. E.

D. P. KRYNINE,† M. AM. SOC. C. E. (by letter).‡—The author's works§ have opened a new era in the study of soils for engineering purposes. Investigations of 10 or 15 years ago generally dealt only with sands, with no consideration for physical properties. Owing to the studies of Professor Terzaghi, the search has penetrated into the interior of clays and sands, the physical properties of which are now being studied by the civil engineer. The present generation is witnessing the birth of a new science, which may be called, "Engineering Soil Science", or "Engineering Soil Theory", and which represents applied soil physics and soil mechanics. It is similar to agricultural soil science; but the latter, by considering principally the influence of water and air on the soil, leaves unstudied the load factor, which is the most important for a civil engineer. Therefore, the establishment of a new science has been inevitable.

Slab Foundations.—In general, the writer agrees with the following statements made by the author:

(1) For soils with great cohesion the settlement produced by a given load increases in direct proportion with the diameter of the loaded area (Fig. 2‖).

(2) The distribution of the soil reactions over the base of a rigid slab is not uniform. The pressures are equal to zero at the edge of the slab and greatest at the center (Fig. 3¶).

(3) The settlement of a building is due to consolidation of the soil and to the lateral flow (Fig. 5**).

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* Discussion of the paper by Charles Terzaghi, M. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Prof. of Highway Eng., Moscow Superior Technical School and Moscow Inst. of Transportation Eng., Moscow, Union of the Socialistic Soviet Republics.

‡ Received by the Secretary, June 23, 1928.

§ "Erdbaumechanik," Vienna, 1925, and others.

‖ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2267.

¶ *Loc cit.*, p. 2270.

** *Loc. cit.*, p. 2272.

However, there are some objections to each of these conclusions.

First.—Attention is called to a paper by Dr. Ing. Koegler,* in which it is stated that the settlement produced by a given load seems to increase directly in proportion with the diameter of the loaded area only to a certain value of this area, after which it becomes more or less constant. In the opinion of Dr. Koegler the elastic condition under a great slab does not permit lateral flow, and the vertical settlement takes place as if the supporting soil were confined laterally. Naturally, the opinion of Dr. Koegler, which seems to be based on his experiments, must be verified.

It also must be noted that his experiments were performed with sands, that is, cohesionless soils. Therefore, the results cannot be generalized until a greater number of experiments with different soils are made; but the example† of the settlement of the Standard Oil Building in San Francisco, Calif., leads the writer to believe in these theories. Under a load of 4 800 lb. per sq. ft., the average settlement of the bearing plate was 0.10 in. The settlement of the building should be $152 \times 0.10 = 15.2$ in.; but the actual settlement was only 2 in. If the soil were homogeneous, this fact must be considered as a proof of Dr. Koegler's statement, but phenomena considered in practice are not as simple and schematic as those studied in a laboratory.

In the example cited the soil was not homogeneous, the settlement of the bearing plate ranging from 0.04 to 0.17 in. The difference is not very great, but in any case there were hard and soft spots in the soil, so that the process of settlement was complicated by the presence of the hard spots which probably produced negative vertical forces acting on the slab. The writer thinks that in the given case and in similar ones it would be better not to deal with the average settlement of the bearing plate, but to elaborate a method of slab design, taking into account the lack of uniformity in the soil.

In regard to the settlement of the caissons‡ at the Chicago Union Terminal, in Chicago, Ill., the writer is not in a position to give a definite opinion on the subject inasmuch as he does not have the description of the work§ at his disposal. Therefore, he does not know how, where, and when the settlement of the caissons was measured. However, he believes that a caisson cannot follow the law of proportionality of the settlement to the diameter of the loaded area because of the friction against the soil.

Let,

H = the height of the circular caisson, in feet (at the Chicago Union Terminal, $H = 60$);

d = the diameter of the circular caisson, in feet;

f = the average frictional force, in pounds per square foot, of the superficial area of the caisson; and,

q = the load, in pounds per square foot.

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* "Die Belastung des Baugrundes," *Der Bauingenieur*, October 27, 1927.

† *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2268.

‡ *Loc. cit.*, p. 2267.

§ *Journal*, Western Soc. of Engrs., February, 1924.

Then, the resultant vertical force, q', would be not q, but:

$$q' = q - \frac{f\,H\,\pi\,d}{\dfrac{\pi\,d^2}{4}} = q - f\,\frac{4\,H}{d} \qquad (22)$$

Equation (22) is an approximate formula which shows that the resultant vertical force acting per unit of the loaded area increases with the increase of the diameter of the caisson. Therefore, the settlement of a large caisson seems to be more considerable than that calculated by Equation (2)* not only because of the direct influence of the diameter, but also because of the great resulting force acting on a unit of loaded ground area in comparison with that acting on a smaller one.

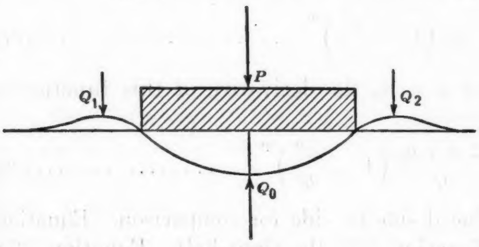

FIG. 38.

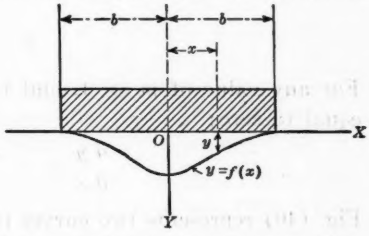

FIG. 39.

Second.—If a slab that is considered uniformly loaded is placed on top of the ground in an excavation, a reaction of the soil develops. Consider only the vertical component of this reaction. The vertical reaction may be confined directly beneath the loaded area (as shown in Fig. 3†); or it may be effective beyond this area (see Fig. 38). In the latter case a condition must be satisfied, such that,

$$P + Q_1 + Q_2 = Q_0 \qquad (23)$$

The case shown in Fig. 38 evidently corresponds to the upheaval produced by the vertical component of the reaction. Since it is admitted that the bulging is due to the lateral flow only,‡ it must be concluded that: $Q_1 = 0$; $Q_2 = 0$. This being the case the derivative of the distribution curve, $y = f\,(x)$, at the edge of the slab, which represents the ratio of an infinitely small increase of the reaction force to the corresponding increase of the abscissa beyond the edges, must be equal to zero:

$$f'\,(b) = 0 \qquad (24)$$

in which, $2b$ is the width of the slab (Fig. 39). As a matter of fact some soil investigations indicate that Equation (24) must be satisfied.§

As to the curve of pressure distribution, the writer's opinion is that it cannot have a parabolic shape for the following two reasons:

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2266.
† *Loc. cit.*, p. 2270.
‡ *Loc. cit.*, p. 2272, and especially Fig. 5.
§ Koegler, "Uber die Verteilung des Bodendruckes unter Gründungskörper," *Der Bauingenieur*, February 5, 1926.

(*a*) The ordinates of a parabola may be calculated accurately according to the equation:

$$y = y_0 \left(1 - \frac{x^2}{b^2}\right) \dots\dots\dots\dots (25)$$

in which, y_0 is the greatest ordinate in the center; x is the distance from the center of the slab; and $2b$ is the width of the slab. The first derivative of y in Equation (25) is:

$$\frac{d\,y}{d\,x} = - y_0 \frac{2\,x}{b^2} \dots\dots\dots\dots (26)$$

which is not equal to zero, when $x = b$.

In order to satisfy the conditions in Equation (24), the shape of the distribution curve may be modified slightly, for instance, so that:

$$y = y_0 \left(1 - \frac{x^2}{b^2}\right)^m \dots\dots\dots\dots (27)$$

For any value of $m > 1$, and for $x = b$, the derivative of this function is equal to zero:

$$\frac{d\,y}{d\,x} = - \frac{2\,m\,x\,y_0}{b^2} \left(1 - \frac{x^2}{b^2}\right)^{m-1} \dots\dots\dots\dots (28)$$

Fig. (40) represents two curves placed side by side for comparison. Equation (25) expresses the left half and Equation (27) the right half. Equation (27) recalls that of the error function, or, better, of the second curve of Pearson.*

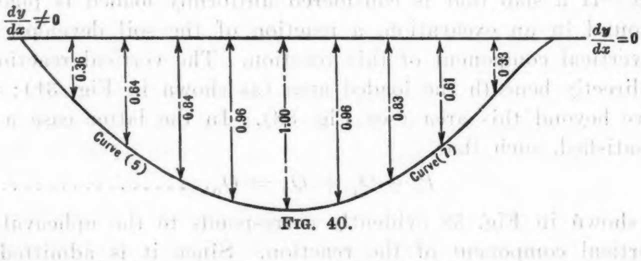

FIG. 40.

(*b*) The area of the curve of pressure distribution, as drawn on the author's Fig. 3 represents the load on a strip of slab 1 ft. wide. If the average pressure on the slab is p_a, it may be written:

$$2\,b\,p_a = \frac{2}{3} \cdot 2\,b\,p_0 \dots\dots\dots\dots (29)$$

and, furthermore,

$$p_0 = \frac{3}{2}\,p_a \dots\dots\dots\dots (30)$$

Equation (30) signifies that the pressure at the center of the slab is always 50% greater than the average. In the writer's opinion the ratio, $\frac{p_0}{p_a}$, depends on the rigidity of the slab. In engineering practice there are no

* *Philosophical Transactions*, Royal Soc. of London, Series A (1895), Vol. 186, Pt. I, pp. 360 and 372; also, Fig. V, p. 365.

infinitely rigid slabs, and, with a decrease in the rigidity of the slab, the ratio $\frac{p_0}{p_a}$, must increase.

The writer insists that in no case does the distribution law follow exactly Equation (27), which has been cited merely as an example.

Third.—The writer believes that the two parallel lines corresponding to two principal sources of settlement (see Fig. 5*) are not really parallel, and that the settlement at the center of a slab is greater than at its edges. For this belief the writer presents the following reasons: (1) In the middle of a building, because of symmetry, there can be no lateral flow; hence, the deflection in the middle of a slab depends only on the vertical component of the pressure; (2) the vertical pressure at the edge being equal to zero, there can be no settlement due to the vertical pressure; hence, the deflection at the edge of a slab depends only on the lateral flow; (3) if the curve of deflection due to vertical pressure (compression) be drawn, it will represent a curve with ordinates decreasing from the center to zero at the edges; on the contrary the ordinates of the curve of deflection due to lateral flow must increase from zero at the center to a certain value at the edges; and (4) if Koegler's theories are right, the lateral flow does not increase from the center toward the edges of the slab during the entire time, but stops when $x < b$ (see Fig. 39). Hence, the settlement at the center of the slab seems to be greater than at the edges. The settlement of small rigid disks, 3 or 5 ft. in diameter, naturally cannot give any idea of the difference of settlement in the center and on the perimeter of the slab.

Generally, a foundation slab is a statically indeterminate structure. The distribution of soil reaction depends on the rigidity of the slab. Therefore, bending moments and shearing forces vary with the change in its rigidity. The distribution of soil reactions is intimately bound to the deflection of the slab, although Hooke's law is not applicable to soils. Hence, the soil deflections, reaction distribution, and stresses in the slab are interrelated. This idea is not new: In 1914, Brugsch and Briske published a paper on the influence of the deflection of the foundation on the design of statically indeterminate structures.†

Entrapped or "Pinched" Air.—Professor Terzaghi greatly merits the approval of the Engineering Profession for demonstrating that clay and water form an intimately connected system. Hence, the mechanics of clay, without considering the influence of water, practically became absurd. The writer desires to show that in some cases not only the water, but also the air, has a considerable influence on the behavior of clay.

The author states‡ that "if the voids of the soil are filled with air, the volume change can take place at once, because the excess air can readily escape toward the surface". The writer thinks the air present in soil may act in three different ways: (1) So that, in one way or another, it com-

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2272.

† Brugsch and Briske, "Einfluss der Nachgiebigkeit des Baugrundes auf die Berechnung äusserlich statisch unbestimmter Bauwerke," *Beton und Eisen*, 1914, pp. 15, 53, 85, and 183.

‡ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2271.

municates to the surrounding atmosphere and assumes directly the temperature and the pressure of the nearest particles of the atmosphere; (2) so that it does not communicate to the atmosphere by means of air channels, but is pressed against soil particles by the water in the soil. This is analogous to a bubble in a spirit-level, and may be termed entrapped or "pinched" air. Such air bubbles may often be microscopic in size; and (3) as adsorbed air which sticks to the particles of soil.

The first of these conditions is the one recognized by Professor Terzaghi. As to the second, which naturally must have great influence on the elastic properties of soils, the writer is not sure of its presence in the deep layers of the ancient natural clays. As to relatively modern deposits, however, and especially the artificial fills there is no doubt as to the presence of entrapped air. Therefore, an investigation of the effect of air and, if necessary, the introduction of the element, "air", into the system, "soil ÷ water", seems to be of definite importance.*

The writer believes that a study of capillarity as applied to water in the soil, leads to the conclusion that there is also a certain quantity of entrapped air.† If the clay pores are not completely filled with water, there must be air present. Furthermore, because clay pores represent excessively thin clefts, it is very difficult to imagine that the air may easily escape from them. At any rate, the influence of the entrapped air on the elastic properties of soils should be investigated.

Pile Foundations.—It is interesting to consider the author's views on pile foundations in reference to the pile-driving formula of Professor Guersevanoff, which is widely used in Russia.‡ Professor Terzaghi's notations§ are used with the following additions:

r = rebound of the hammer after the blow; and

a = coefficient of the lost work, depending on the nature of the pile, method of its driving, etc.

The work done by the drop of a hammer consists of three parts: (1) Pile penetration; (2) elastic deformation; and (3) lost work; for instance, increase in temperature, etc. This may be expressed:

$$R\,h = [A] + [B] + [C] = Q_a\,s + R\,r + a\,R\,h \qquad (31)$$

In his analysis, Guersevanoff ignores the term, $R\,r$, so that Equation (31) becomes:

$$(1 - a)\,R\,h = Q_d\,s \qquad (32)$$

The coefficient, $1 - a = b$, depends on the value of Q_d and the cross-section of the pile, F; that is,

$$b = f\left(\frac{Q_d}{F}\right) \qquad (33)$$

* D. P. Krynine, "On the Technical Rôle of Air in Soils" (in Russian), *Transactions*, Inst. of Structural Research, 1928.

† D. P. Krynine, M. V. Ivanova, and T. A. Ovsiannikoff, "On the Capillary Rise" (in Russian), *Transactions*, Inst. of Structural Research, 1928.

‡ *The Cement* (Russian Magazine), 1916; First issue, p. 2; Second issue, p. 73. This magazine is no longer published.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2273–2274

By making various assumptions he comes to the conclusion that the shape of the curve of Equation (31) is a hyperbolic one:

$$b = \frac{R + m^2 G}{R + G} \times \frac{1}{1 + \frac{Q_a}{F} n} \quad \text{.................. (34)}$$

The coefficient, n, depends both on the nature of the pile and on the method of its driving. For instance, for wooden piles without followers, $n = 10 \frac{K g}{c m^2}$; for concrete piles with followers, $n = 0.5 \frac{K g}{c m^2}$. Definitely, the formula, in kilograms and centimeters, is:

$$Q_a = -\frac{n}{2} + \sqrt{\frac{n^2}{4} F + n \frac{F}{s} R h \frac{R + 0.2\, G}{R + G}} \quad \text{..........(35)}$$

This formula has somewhat the appearance of Equation (3),* but Equation (35) neglects partly the length of the pile, and the modulus of elasticity of the pile material enters implicitly into the value of n.

The author's concept of hydrodynamic stresses induced in soils is truly brilliant, but there is some question as to the squeezing of water out of soil beneath the point of the pile.† Suppose the pile is driven into clay containing 50% of voids which are completely filled with water. Then, the thickness of the film of water around the pile must be, approximately (deformations of the pile disregarded):

$$T = \frac{0.5 \frac{\pi d^2}{4} h}{\pi d h} = \frac{d}{8}$$

in which, d is the diameter of the pile. In Fig. 41 the full line represents the position of the pile before the blow and the dotted line, the position after the blow. Thus, the film of water around a 10-in. pile would be $1\frac{1}{4}$ in. thick, which seems to be too much. Therefore, the writer thinks that the vertical force transmitted by the pile: (*a*) Compresses the soil, thereby causing the pile to move downward and producing a certain lateral flow of hard particles; and (*b*) acts on the soil water just as in any case of clay loading.‡ In Fig. 42 the part of the vertical force acting on the water is divided into two components: (1) The horizontal force, moving water away from the pile and thus compressing the entrapped air; and (2) the force parallel to the surface of the pile. The reaction against this component of force is what causes the film to form on the surface of the pile. The posterior absorption of the film by the soil may occur, in the writer's opinion, only if there is air in the voids. The layers of clay in Fig. 42 are assumed to be approximately horizontal.§

Russian Contribution to the Science of Foundations.—The author deals with the present status of the science of foundations as well as with its future.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2274.

† *Loc. cit.*, pp. 2278–2279.

‡ Terzaghi, "Erdbaumechanik," Abschnitt 12, 20, and others.

§ D. P. Krynine, "Elementary Proof of Shale-Likeness of Clay Particles," *Public Roads*, January, 1928.

Russian engineers have made some noteworthy contributions to the theory of pile-driving.* They have also published some works dealing with the determination of the depth, t, of the foundation. Through a paper by Professor Kjürdümoff† three formulas, those of Paucker and Jankowsky, came from Russia into European literature.‡

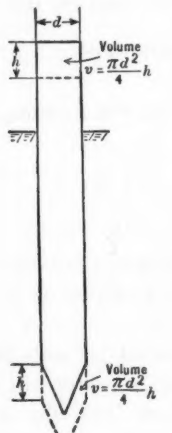

FIG. 41.

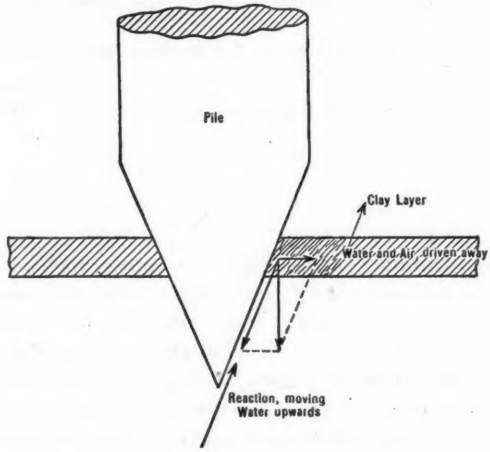

FIG. 42.

Let,

t = the depth of the foundation;

H = the height of a sand prism, with base area and weight equal to those of the building; and,

ϕ = the angle of the internal friction.

Then, Paucker's formula is:

$$t = H\, t\, g^4 \left(45^\circ - \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

The first formula of Jankowsky is:

$$t = \frac{H}{2}\, t\, g^4 \left(45^\circ - \frac{\phi}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

and his second formula is:

$$H = 2\, t \frac{t\, g^2\left(\frac{45^\circ + \phi}{2}\right)}{t\, g^2\left(\frac{45^\circ - \phi}{2}\right)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

* Jankowsky, "On the Resistance of Pile Foundations," *Journal*, Ministry of Ways of Communication, 1887; Guersevanoff, the previously cited paper, 1916; Dmohovsky, Papers on the influence (a) of the geometric shape of piles; and (b) of the eccentricities of loads, Works of the Moscow Inst. of Transportation Eng., 1927. There are also other papers by the same author and his textbook on "Engineering Foundations," 1928.

† Kjürdümoff, "Zur Frage des Widerstandes der Gründungen auf natürlichen Böden," *Der Civilingenieur* 1892.

‡ The derivation of these formulas is to be found in Terzaghi's "Erdbaumechanik," 1925, p. 229.

Belzetsky's formula* is:

$$t = H\,t\,g^4\left(45^\circ - \frac{\phi}{2}\right) - b\,\frac{1 - t\,g^4\left(45^\circ - \frac{\phi}{2}\right)}{2\,t\,g\left(45^\circ - \frac{\phi}{2}\right)} \quad \text{.........(39)}$$

He considers the effect of a horizontal force as well and admits that the soil reactions follow the sine law.† The same problem has been outlined by Prokofieff.‡ The writer is also occupied at present with the experimental study of this problem.

Miniaeff§ proposed the formula:

$$t = \left(\frac{1+n}{n_1 - n} \cdot \frac{\sin\alpha}{\pi} - \frac{1-n}{n_1 - n} \cdot \frac{\alpha}{\pi}\right) H - \frac{b}{2}\,C\,t\,g\,\frac{\alpha}{2} \quad \text{........(40)}$$

in which,

$$\cos\alpha = \frac{1}{1+n} - \sqrt{\frac{n^2}{(1+n)^2} + \frac{(n_1 - n)\,\pi\,\alpha}{2\,(1+n)\,H}}$$

$$n_1 = t\,g^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

and,

$$n = t\,g^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right)$$

Equation (40) is based on the following hypotheses: (1) The distribution of stresses in soils follows the same laws as in a solid; (2) "sets" in a dry substance are not functions of forces; and (3) stresses in dry substance are determined only by elastic deformation. The problem consists in determining if, under a given set of forces, an elastic deformation of the soil takes place. If not, a supplementary set of forces must be applied, namely, the weight of the soil above the level of the bottom of the foundation.

Pouzyrewsky‖ considered a condition of the soil when all the deformations are elastic only, and proposed the formula:

$$t = H\,\frac{C\,t\,g\,\phi + \phi - \frac{\pi}{2}}{C\,t\,g\,\phi + \phi + \frac{\pi}{2}} \quad \text{.......................(41)}$$

In his paper on the theory of dry substances, Nemiloff¶ developed a formula which, like the formula of Belzetsky, takes into account the width, *b*, of the foundation. It is worthy of note that, in 1913, he proposed an apparatus for determining the coefficient of internal friction** based on practically the same idea as that for the apparatus used in 1917 by the Society's Special

* Belzetsky, "Statics of Constructions," 1914, p. 95.

† *Loc. cit.*, p. 83.

‡ Prokofieff, "Theory of Constructions," 1928, Vol. II, p. 258.

§ Miniaeff "On the Distribution of Stresses in Dry Substances," Tomsk, Siberia, 1915, p. 85.

‖ Pouzyrewsky, "Foundations Design" (in Russian), 1923, p. 38.

¶ Nemiloff, "Contribution to the Theory of Dry Substances," *Journal*, Ministry of Ways of Communications, 1913, Issue 9, pp. 126–148.

** *Loc. cit.*, p. 140, Fig. 3.

Committee to Codify Present Practice of the Bearing Power of Soils for Foundations, etc.*

This fact shows that there is a need for a scientific center to look after the technical literature in civil engineering the world over, in order to avoid the duplication of work and to utilize the technical ideas of different nations. The Society seems to be an appropriate agency for this purpose.

Generally, Russian civil engineers seem to like the soil and dry substances theory, and there are many papers and discussions dedicated to this subject. The writer calls attention to the works of Prilejaeff† and Spalving.‡ The latter gives a new theory; he not only takes into account the equilibrium of a soil prism as Paucker and others do, but he studies the stresses that develop in the interior of this prism as well.

Since 1922, Russian engineers (principally highway engineers) have given their attention to the necessity of studying the physical properties of soils. The writer was the first in Russia who expressed in printed form the necessity of co-operation between highway engineers and soil scientists.§ In 1923, the Russian Highway Department and, later, its Highway Research Bureau, began its activities along this line. When Professor Terzaghi's "Erdbaumechanik" appeared in 1925, it became clear that a new "slant" had been given to this science.

Soil Classification.—Engineering soil science has advanced considerably in recent years; but a definable system of soil classification has not appeared until now. The research engineer is in a difficult position when receiving a sample of soil from a job. He may tell the builder as much as he wishes about the internal friction of the soil, about its compressibility, its permeability, etc.; but the only question asked is, "How many pounds pressure will this soil support?"

A laboratory cannot produce the answer to this question categorically. The most practical solution of the problem seems to be to elaborate a combination of simple laboratory experiments with a simple routine test in the field.

* *Proceedings*, Am. Soc. C. E., August, 1917, Papers and Discussions, p. 1179; and, further, Fig. 1, p. 1174.

† Prilejaeff, "Contribution to the Theory of Pressure on Sustaining Walls" (in Russian), 1913.

‡ Spalving, "A New Theory of Dry Substances," *Water Transport* (Russian magazine), February, 1927.

§ D. P. Krynine, "American Methods of Earth-Road Building and Their Application in Russia," *The Messenger of Technics and Economics* (Russian magazine no longer published), January, 1922.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

Discussion*

BY MESSRS. FREDRIK VOGT AND A. FLORIS

DR. FREDRIK VOGT† (by letter).‡—The author deserves credit for having brought this method of calculation to discussion, since it may lead to improvements in the design of dams. Their main discussion is confined to the calculation of stresses under certain conditions, and the assumptions made are only partly dealt with. It seems to the writer that a discussion of the assumptions is the more important phase. A rough approximation based on correct assumptions with regard to shrinkage, temperature changes, yielding of foundation, etc., is more valuable than a highly refined computation based on wrong assumptions. However, it is easily understood that the authors wished to limit the problem.

Ignoring all parts of the structure in which tensile stresses occur in order to calculate the greatest compression stresses seems at first glance to be on the side of safety. However, if at first a single crack is opened, due to high tension, the tensile stresses at the neighboring sections are at once reduced to such small magnitudes, that the occurrence of more cracks is not probable. A cantilever with only one horizontal crack at the foundation joint is less flexible than a cantilever without any tensile strength.§ In order to obtain the necessary deflection of the arch rings, the eccentricity of the load on the cantilevers and the compression stresses at the toe must be greater if only a single crack is formed than if all the tensile strength can be neglected.

When a large part of the arch section is either affected by tensile stresses or is cracked, the material is only partly utilized. However, when the dam is constructed in the usual manner, it is impossible to avoid this waste of mate-

* Discussion of the paper by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., continued from October, 1928, *Proceedings*.

† Civ. Engr., Trondhjem, Norway; temporarily with U. S. Bureau of Reclamation on Arch Dam Model Investigations, Boulder, Colo.

‡ Received by the Secretary, May 29, 1928.

§ Some experiments on this subject have been made by E. Aarskog, Civ. Engr., Dept. of Public Roads, Oslo, Norway.

rial. The writer has worked on the problem of how to make the cylinder formula valid for arch dams, thus adding to the ability of the arches to take loads without increasing the stresses in order to improve the safety factor or to cut down the dimensions of the structure. He thinks that this result can be obtained by using a multiple-pressure grouting system, dividing a number of the contraction joints into closed compartments by vertical and horizontal water-stops, so that each of the compartments can be grouted under separate pressure. By such a system it is possible to apply initial stresses in the dam, distributed in such a way that the resulting stresses due to load, shrinkage, cooling, and grouting are approximately equalized over the section. Not only will the arch effect be improved by making the cylinder formula valid in this way, but by pressing grout into closed compartments an actual pressure grouting also can be obtained in order to reduce the stresses in the cantilevers and at least to some extent convert the tensile stresses into compression. The writer agrees with the authors that dams constructed as at present, without any actual pressure grouting, should be designed with constant radius where the canyon is wide as compared with the height of the dam.* However, by applying a multiple-pressure grouting, the use of the constant-angle arch principle can also be made suitable for dams in wide canyons, since the cantilever load can be checked. Furthermore, the use of partly open joints of, say, ½-in. to 1-in. opening, makes it possible to cool the dam by circulating water in order to get rid of the chemical heat before grouting and to apply the grouting without waiting for a natural opening of the joints due to shrinkage and cooling.

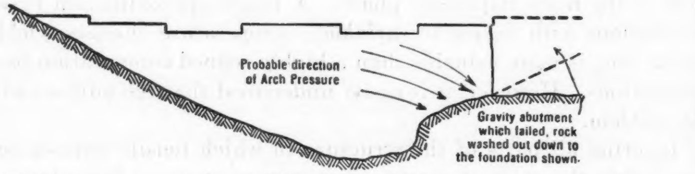

FIG. 37.

The method of calculation in itself does not seem quite adequate until three factors affecting the load distribution are taken into account: First, the bending of the cantilevers due to the lateral expansion which is caused by arch stresses and *vice versa;* second, the effect of stiffness against torsion; and, third, the effect of stiffness against tangential displacements in the structure.

At the crown, the water-load produces high compression stresses at the up-stream face and low compression on tensile stresses at the down-stream face. The lateral expansion due to these stresses tends to bend the cantilever down stream, thus reducing the bending stresses in the cantilevers. After the arch stresses are computed, this bending can easily be calculated for all cantilevers. The influence of lateral expansion—due to stresses in the cantilevers from water-load only—on the arch deflection will usually be negligible.

In two cases the writer has found that the stiffness against torsion of cantilevers and arch rings reduces the stresses slightly, but, since this effect is of

* *Proceedings,* Am. Soc. C. E., January, 1928, Papers and Discussions, p. 76.

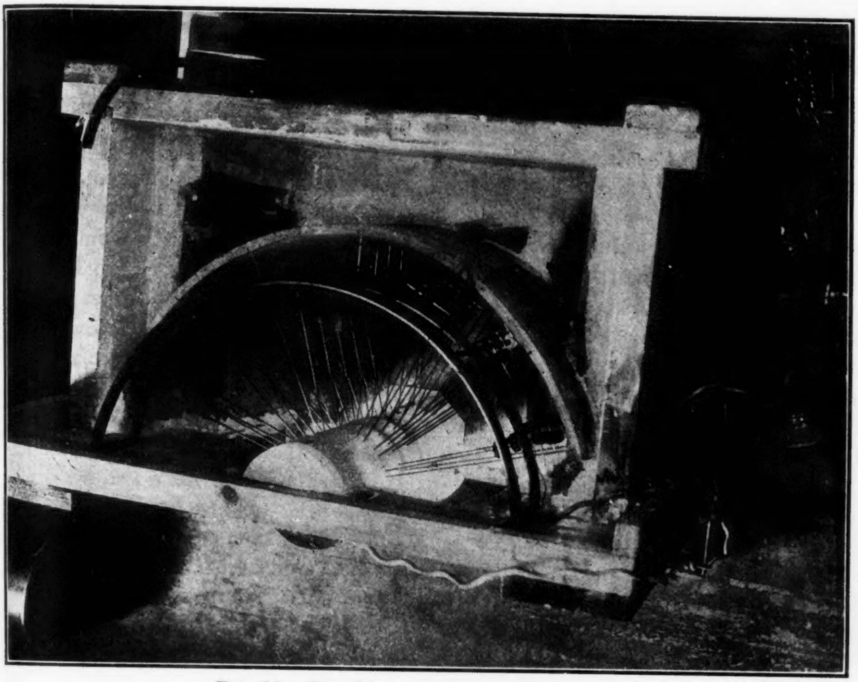

FIG. 38.—TOP VIEW OF RUBBER MODEL DAM.

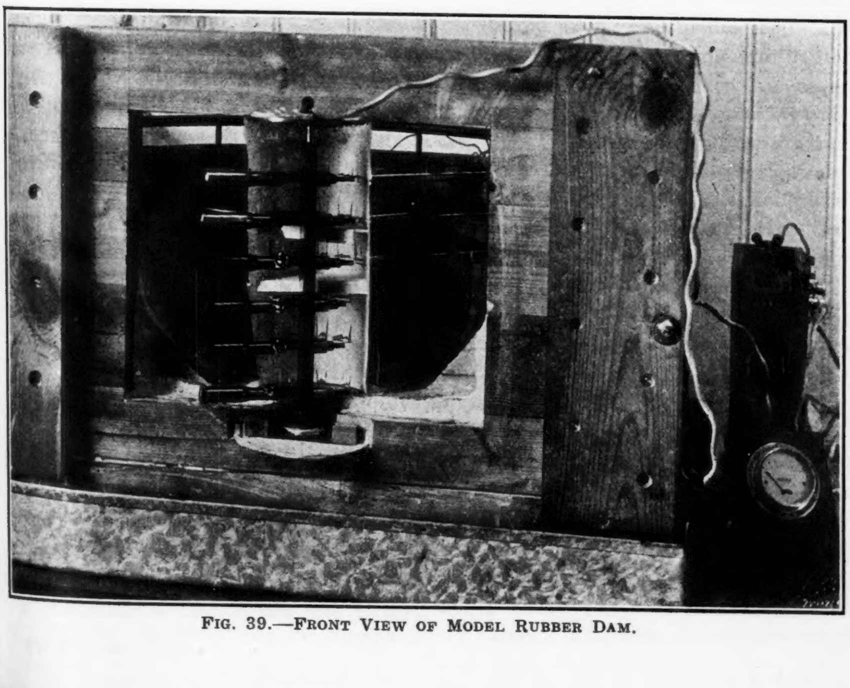

FIG. 39.—FRONT VIEW OF MODEL RUBBER DAM.

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little importance, at least for the examples dealt with, and as they are difficult to compute, he agrees that it is best to neglect it. Perhaps, the torsion may be of importance for very thick arch dams.

The stiffness against tangential displacements, on the other hand, may be of considerable importance in some cases, and when taken into account may cause a considerable redistribution of the stresses. The story of two dam failures described in the technical press* gives a convincing illustration of the importance of this stiffness. In one case the rock abutment of the arch was washed out; in the other case a gravity abutment turned over, so that both arches were without support for more than one-half their height. However, the arch itself did not fail in either case. If the dams were considered as consisting of arch rings and cantilevers, as discussed by the authors, and the arch rings were without support and were unable to take any load in the upper half of the height, then the cantilevers would be certain to fail since the sections were very slender. In the second case, the dam would have to stand the total water pressure without support from the abutment for some time before the water was lowered. Moreover, it would have to stand the shaking due to the failure of the abutment. Only factors other than those dealt with in the paper could save the arches, and among these the stiffness against the tangential displacements may be the most important. It also can be stated that the arch pressure in this case has been directed down to the remaining foundation, as indicated in Fig. 37.

In order to check the importance of this stiffness, in the case of dams with abutments which have not failed, the writer made tests on a small rubber model in Trondjhem, Norway. (See Figs. 38 and 39.) The model itself and the foundation were both of soft rubber mounted in a wooden frame and loaded with water. Radial and tangential displacements were measured for seven vertical and six horizontal sections (see Fig. 40). The diagrams show at once that there is a twisting at the foundation due to a bending moment which verifies the calculations made by the writer† for the yielding of elastic foundations. Furthermore, tangential displacements are found to have about the expected magnitude, and they cannot occur except in connection with tangential shear forces, which tend to increase the stiffness of the arch rings against deflection. All the different kinds of rubber considered showed a high time effect and did not give an entirely straight stress-strain diagram. Furthermore, Poisson's ratio was about 1:2.2 for the rubber chosen for the experiment, instead of 1:6 or 1:8 for concrete, and this makes some difference in the coefficients for the shear deformation.

Due to the time effect, it was difficult to get accurate measurements, and these may be taken mainly as qualitative and not quantitative. The radial deflection was measured directly by means of micrometer screws with electric contacts. The tangential displacements were converted into radial displacements by means of a lever system, and were measured by means of the same screws. These screws were mounted on a turnable post, one screw at each eleva-

* *Engineering News-Record,* Vol. 2, 1926, p. 616.

† "Ueber die Berechnung der Fundamentdeformation," *Det Norske Videnskapsakademi.* Oslo, Norway, 1925.

tion for which the displacements were to be measured. The observations were read to the nearest 0.0002 in. and the maximum deflection was about 0.06 in. The stiffness against tangential displacements is of importance mainly at dams with a great ratio between span and height, that is, for all the dams referred to in the paper.

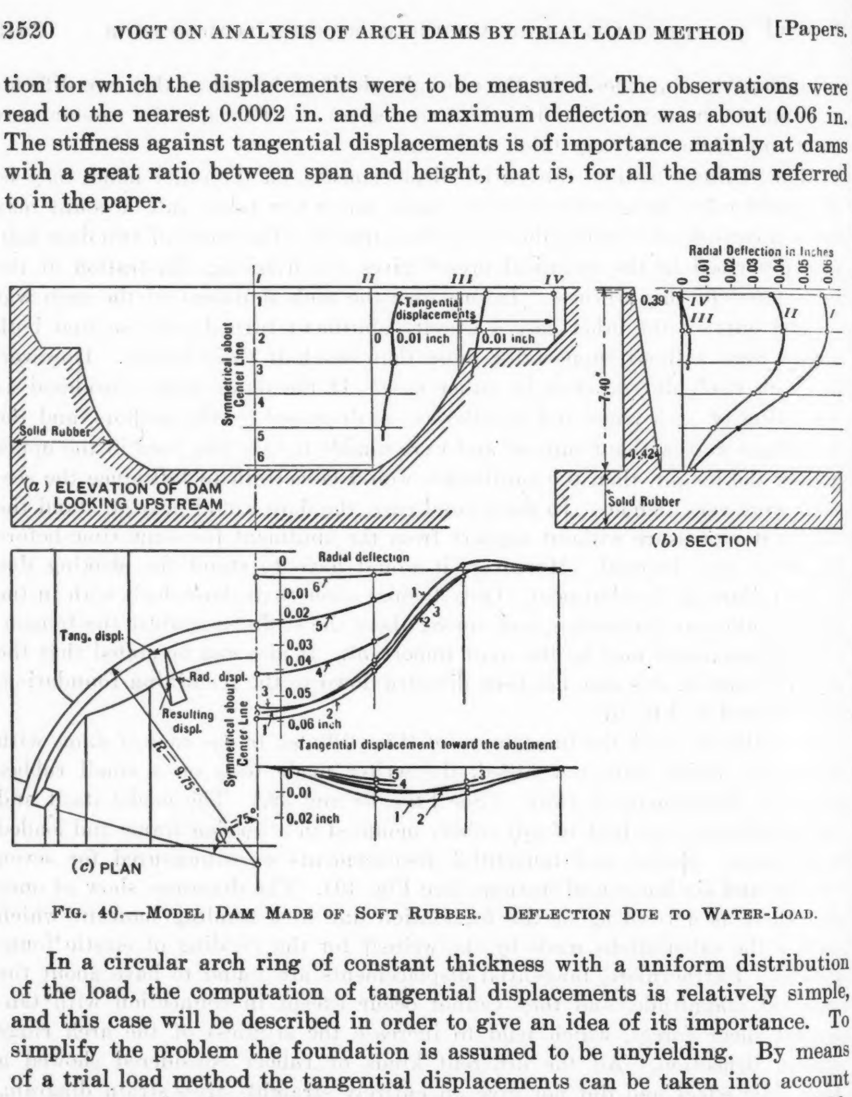

FIG. 40.—MODEL DAM MADE OF SOFT RUBBER. DEFLECTION DUE TO WATER-LOAD.

In a circular arch ring of constant thickness with a uniform distribution of the load, the computation of tangential displacements is relatively simple, and this case will be described in order to give an idea of its importance. To simplify the problem the foundation is assumed to be unyielding. By means of a trial load method the tangential displacements can be taken into account more accurately whether or not the foundation is assumed to be firm or yielding.* The matter of making the computations is mainly a question of patience and time.

The method of computing the radial deflection in arches is well known and the same method can also be applied to the calculation of the tangential displacements. Therefore, the writer gives the final formulas without derivations.

* See the writer's discussion of the paper by B. F. Jakobsen, M. Am. Soc. C. E., entitled "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475, and the Report on the Stevenson Creek Test Dam, *Proceedings*, Pt. 3, Am. Soc. C. E., May, 1928, p. 129, where it is stated that the writer's formulas for the foundation yielding are checked by the experiments. These formulas are also included in the trial load method as used at present by the U. S. Bureau of Reclamation, see the discussion by J. L. Savage and Ivan E. Houk, Members, Am. Soc. C. E., on this paper, *Proceedings*, Am. Soc. C. E., October, 1928, Papers and Discussions, p. 2317.

The radial deflection at the crown of the arch due to a uniform load, p_0 (Fig. 41), is:

$$\Delta r_0 = \lambda_0 \, \rho \, \frac{p_0 \, r_n \, r_e}{E \, t} \qquad \text{(115)}$$

in which, λ_0 and ρ are two coefficients;* r_n = the radius to the neutral axis; r_e = the radius to the up-stream face; t = the thickness of the arch; and E = the modulus of elasticity.

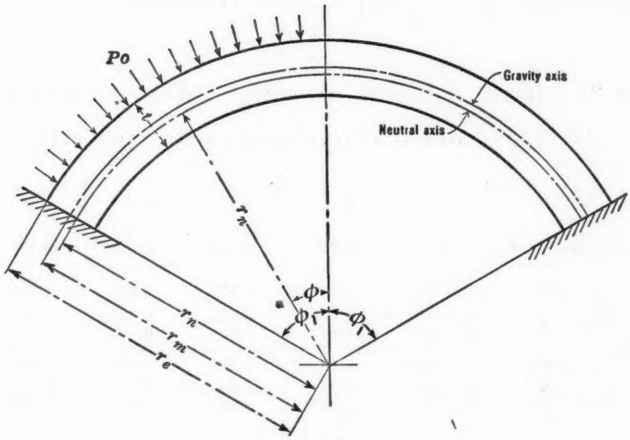

FIG. 41.

The radial deflection, Δr, at any point of the arch compared with the deflection at the crown is given by Equation (116):

$$\frac{\Delta r}{\Delta r_0} = \frac{a\,[\phi_1\,(1 - \cos\phi_1 \cos\phi) - \phi \sin\phi \sin\phi_1] - b\,[\cos\phi - \cos\phi_1] \sin\phi_1}{(1 - \cos\phi_1)\,(a\,\phi_1 - b \sin\phi_1)} \qquad \text{(116)}$$

in which,

$$a = 1 + \frac{1}{3}\left(\frac{\kappa E}{G} + 1\right)\left(\frac{t}{2\,r_n}\right)^2$$

and,

$$b = 1 - \frac{1}{3}\left(\frac{\kappa E}{G} - 1\right)\left(\frac{t}{2\,r_n}\right)^2$$

Equation (116) equals approximately:

$$\frac{\Delta r}{\Delta r_0} = \left[1 - \left(\frac{\phi}{\phi_1}\right)^2\right]^2 \qquad \text{(117)}$$

in which, ϕ_1 is one-half the central angle, and ϕ, the angular distance from the crown to the point in question, both in radians.

Furthermore, at any point except the crown and the abutments a tangential displacement, Δb, directed toward the abutments takes place, as follows:

* See the writer's discussion on the paper by B. F. Jakobsen, M. Am. Soc. C. E., entitled "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 544, Equations (120), p. 558, and (125), p. 564, and Fig. 46, p. 565.

$$\frac{\varDelta b}{\varDelta r_0} = \frac{a\,[\phi \cos \phi \sin \phi_1 - \phi_1 \cos \phi_1 \sin \phi] - 2\,\dfrac{\sin \phi_1}{\phi_1}\,[\phi_1 \sin \phi - \phi \sin \phi_1]}{(1 - \cos \phi_1)\,(a\,\phi_1 - b \sin \phi_1)} \quad \ldots (118)$$

or, approximately,

$$\frac{\varDelta b}{\varDelta r_0} = \frac{\phi}{15}\left[7 - 10\left(\frac{\phi}{\phi_1}\right)^2 + 3\left(\frac{\phi}{\phi_1}\right)^4\right] \quad \ldots\ldots\ldots\ldots (119)$$

The accuracy of Equations (117) and (119) is illustrated by Tables 14 and 15. At the quarter-points, $\left(\phi = \dfrac{1}{2}\,\phi_1\right)$, Equation (119) gives $\varDelta b = \dfrac{5}{32}\,\phi_1\,\varDelta r_0$.

TABLE 14.—RATIO, $\dfrac{\varDelta r}{\varDelta r_0}$, FOR DIFFERENT VALUES OF THE RATIO, $\dfrac{\phi}{\phi_1}$, ACCORDING TO EQUATION (116) AND EQUATION (117).

$\frac{\phi}{\phi_1} =$			0	0.25	0.5	0.75	1.00
Approximate Equation (117)...			1.00	0.879	0.563	0.191	0.00
Exact Equation (116)	$\frac{t}{2\,r_n} = 0$	$\phi_1 = 90°$	1.00	0.870	0.540	0.175	0.00
	$= 0$	$\phi_1 = 60°$	1.00	0.875	0.552	0.184	0.00
	$= 0$	$\phi_1 = 30°$	1.00	0.879	0.559	0.190	0.00
	$\frac{t}{2\,r_n} = 0.1$	$\phi_1 = 90°$	1.00	0.872	0.545	0.181	0.00
	$= 0.1$	$\phi_1 = 60°$	1.00	0.880	0.569	0.202	0.00
	$= 0.1$	$\phi_1 = 30°$	1.00	0.895	0.611	0.258	0.00

TABLE 15.—RATIO, $\dfrac{\varDelta b}{\varDelta r_0}$, FOR DIFFERENT VALUES OF THE RATIO, $\dfrac{\phi}{\phi_1}$, ACCORDING TO EQUATION (118) AND EQUATION (119).

	$\frac{\phi}{\phi_1} =$		0	0.25	0.50	0.75	1.00
$\frac{t}{2\,r_n} = 0$	$\phi_1 = 90°$	Exact.........	0	0.171	0.247	0.181	0
		Approximate..	0	0.167	0.245	0.183	0
	$\phi_1 = 60°$	Exact.........	0	0.113	0.164	0.121	0
		Approximate..	0	0.111	0.164	0.122	0
	$\phi_1 = 30°$	Exact.........	0	0.054	0.082	0.062	0
		Approximate..	0	0.056	0.082	0.061	0
$\frac{t}{2\,r_n} = 0.1$	$\phi_1 = 90°$	Exact.........	0	0.171	0.248	0.182	0
		Approximate..	0	0.167	0.245	0.183	0
	$\phi_1 = 60°$	Exact.........	0	0.113	0.166	0.124	0
		Approximate..	0	0.111	0.164	0.122	0
	$\phi_1 = 30°$	Exact.........	0	0.054	0.084	0.067	0
		Approximate..	0	0.056	0.082	0.061	0

For instance, if one-half the central angle, ϕ_1, is equal to 1.0472 radians (60°), the tangential displacement midway between the crown and the abutment is found to be about one-sixth the radial deflection at the center.

If the dam were divided by contraction joints very closely spaced, and if there were no friction in the joints, there would be no resistance against such

tangential displacements, since each section would then be very flexible in this direction. However, the space between the contraction joints for most dams (very large dams excepted) is usually greater than the thickness of the dam, and hence the stiffness against tangential displacements of each section taken separately is greater than against radial deflections. Then, too, there is always a large amount of friction (due to the arch thrust) to resist the sliding of the two sections relative to each other. Especially if the joints are grouted, it must be assumed that this friction is great enough to make the dam act as a monolithic structure. If the sections can slide relative to each other, a tangential displacement can be produced by the bending of the sections, as shown in Fig. 42(*a*); but if the sections cannot slide relatively, the tangential displacement can be produced only as a shear deformation, each square being limited by vertical radial planes and horizontal planes must be deformed as shown in Fig. 42(*b*). This diagram also shows the shear stresses producing the deformation. If the bending deformation is taken away, and only the shear deformation remains, a relatively large force is needed to produce a certain displacement. In other words, such a condition makes a dam very resistant to tangential displacements.

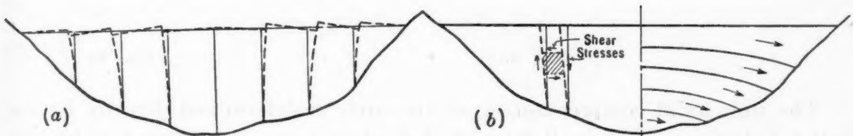

FIG. 42.

If this shear is combined with the arch thrust, the direction of the principal stresses may be about as shown on the right side of Fig. 42(*b*); the arch thrust is bent downward in an inclined direction at the abutments. If the abutment has a salient angle, as in the case of the rubber model, the thrust is concentrated just at that point, as may be seen from the great tangential displacement (yielding foundation) in Fig. 40.

Since, in the two dams mentioned, the abutments failed and the arches did not, it may be possible to construct safe multiple-arch dams without buttresses, as shown in Fig. 43; the arch thrust at the center of the dam is led down to the foundation at the end of the vertical arches, and the arches are connected in such a way that the water pressure not balanced by arch thrust or cantilever action is balanced by tensile stresses at the reinforced connection between the arches.

In this connection, it is of value to compute the deflection of the arch ring if all tangential displacements absolutely are prevented and the arch is loaded with a uniformly distributed load and other conditions are not changed. Besides the normal force, N, the shear, S, and the moment, M, at both ends of an element of the arch, a tangential force, Q, per unit length of the arch axis, is needed to prevent these tangential displacements (Fig. 44). This force, Q, is the resultant of the tangential shear at the upper and lower boundary of the arch ring. The thickness measured square on the plane of Fig. 44 is assumed

to be equal to the unit. Three conditions are necessary in order that the hatched element in Fig. 44 shall be in equilibrium:

$$d N + Q r_n d \phi = 0 \qquad (120)$$

$$N d \phi + d S - p_1 r_e d \phi = 0 \qquad (121)$$

$$S r_n d \phi - d M = 0 \qquad (122)$$

The total radial deflection, y, can be considered as consisting of the deflection, y_1, due to the moment, M, and y_2 due to the shear, S:

$$y = y_1 + y_2 \qquad (123)$$

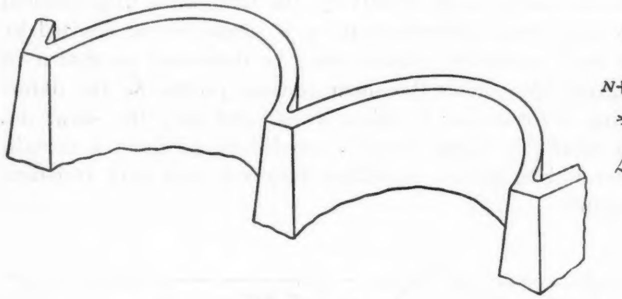

FIG. 43.

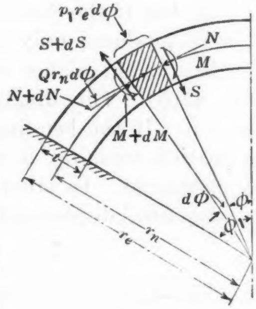

FIG. 44.

The unit axial compression, u, of the arch is determined directly by the radial deflection, y, since all tangential displacements are assumed to be prevented and only radial deflections are allowed:

$$u = \frac{y}{r_n} \qquad (124)$$

Furthermore, the axial compression can be expressed by the normal force:

$$N = u E \frac{t}{\rho} = y E \frac{t}{\rho r_n} \qquad (125)^*$$

The usual formulas give the derivatives of the terms, y_1 and y_2, which together constitute the total deflection:

$$\frac{d^2 y_1}{r_n{}^2 d \phi^2} = \frac{M}{E I} = \frac{12 M}{E t^3} \qquad (126)\dagger$$

$$\frac{d y_2}{r_n d \phi} = -\frac{\kappa S}{G F} = -\frac{\kappa S}{G t} \qquad (127)$$

By using the notations,

$$y_1 = \rho \frac{p_1 r_e r_n}{E t};\ a^4 = \frac{3 r_n{}^2}{\rho t^2};\ b^2 = \frac{1}{4 \rho} \frac{\kappa E}{G} \qquad (128)$$

* ρ is equal to unity for "thin" arches.

† For thick arches the moment of inertia, $I = \frac{t^3}{12}$, used as in Equation (126) does not give absolutely exact values for the bending, but the difference is without any importance, as shown by B. F. Jakobsen, M. Am. Soc. C. E., in his paper, "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 475.

an elimination of the different unknown magnitudes by means of Equations (120) to (128) gives the differential equation for the deflection,

$$\frac{d^4\,y}{d\,\phi^4} - 4\,b^2\,\frac{d^2\,y}{d\,\phi^2} + 4\,a^4\,y = 4\,a^4\,y_1 \quad \text{(129)}$$

The moment and shear can be expressed by the deflection:

$$M = \frac{E\,t}{4\,a^4\,\rho}\left[\frac{d^2\,y}{d\,\phi^2} - 4\,b^2\,(y - y_1)\right] \quad \text{(130)}$$

$$S = \frac{E\,t}{4\,r_n\,a^4\,\rho}\left[\frac{d^3\,y}{d\,\phi^3} - 4\,b^2\,\frac{d\,y}{d\,\phi}\right] \quad \text{(131)}$$

From the symmetry of the arch it follows that at the crown ($\phi = 0$):

$$\frac{d\,y}{d\,\phi} = \frac{d^3\,y}{d\,\phi^3} = 0 \quad \text{(132)}$$

At the abutments ($\phi = \phi_1$), $y = 0$. Furthermore, the moment in itself produces no change in direction for the arch axis at the abutment if the dam is assumed to be fixed at both ends. The change in direction there can only be caused by the shear, so that for $\phi = \phi_1$:

$$\frac{d\,y}{d\,\phi} = \frac{d\,y_2}{d\,\phi} = -\frac{\kappa\,r_n}{G\,t}\,S = -\frac{b^2}{a^4}\left[\frac{d^3\,y}{d\,\phi^3} - 4\,b^2\,\frac{d\,y}{d\,\phi}\right] \quad \text{(133)}$$

or,

$$b^2\,\frac{d^3\,y}{d\,\phi^3} + (a^4 - 4\,b^4)\,\frac{d\,y}{d\,\phi} = 0 \quad \text{(134)}$$

The general differential equation is satisfied by the expression:

$$y = y_1 + C_1\,e^{\alpha\phi}\sin\beta\,\phi + C_2\,e^{\alpha\phi}\cos\beta\,\phi + C_3\,e^{-\alpha\phi}\sin\beta\,\phi + C_4\,e^{-\alpha\phi}\cos\beta\,\phi \quad \text{(135)}$$

in which,

$$\alpha = \sqrt{a^2 + b^2} \text{ and } \beta = \sqrt{a^2 - b^2} \quad \text{(136)}$$

and C_1 and C_4 are four integration constants. These constants must be determined by the conditions at $\phi = 0$, and $\phi = \phi_1$, previously discussed. The single solution which agrees with these conditions is:

$$y = y_1\,[1 - A\sinh\alpha\,\phi\sin\beta\,\phi - B\cosh\alpha\,\phi\cos\beta\,\phi] \quad \text{(137)}$$

in which,

$$A = \frac{\alpha(3\,\beta^2 - \alpha^2)\sinh\alpha\,\phi_1\cos\beta\,\phi_1 - \beta\,(3\,\alpha^2 - \beta^2)\cosh\alpha\,\phi_1\sin\beta\,\phi_1}{\beta\,(3\,\alpha^2 - \beta^2)\sinh\alpha\,\phi_1\cosh\alpha\,\phi_1 + 2\,(3\,\beta^2 - \alpha^2)\sin\beta\,\phi_1\cos\beta\,\phi_1} \quad \text{(138)}$$

and,

$$B = \frac{\beta(3\,\alpha^2 - \beta^2)\sinh\alpha\,\phi_1\cos\beta\,\phi_1 + \alpha\,(3\,\beta^2 - \alpha^2)\cosh\alpha\,\phi_1\sin\beta\,\phi_1}{\beta(3\,\alpha^2 - \beta^2)\sinh\alpha\,\phi_1\cosh\alpha\,\phi_1 + \alpha\,(3\,\beta^2 - \alpha^2)\sin\beta\,\phi_1\cos\beta\,\phi_1} \quad \text{(139)}$$

At the crest of the arch, $\phi = 0$, the deflection is found from Equation (137):

$$\Delta\,r_1 = y_1\,[1 - B] = \lambda_1\,\rho\,\frac{p_1\,r_n\,r_e}{E\,t} \quad \text{(140)}$$

in which,

$$\lambda_1 = 1 - B \quad \text{(141)}$$

and y_1 is the deflection according to the cylinder formula.

As may be seen from this development, the final formula (Equation (140)) for the deflection at the crown of the arch is the same whether or not the arch ring is prevented from tangential displacements, except that the coefficient, λ_0, in Equation (115) must be replaced by λ_1, if these displacements are prevented. By calculating the coefficients, λ_0 and λ_1, the influence of the stiffness against tangential displacements on the stiffness against radial displacements can be judged. For instance, for an arch with a thickness, t, equal to one-twentieth the radius, r_n, of the neutral axis, Equation (115) shows that if $\frac{\kappa E}{G} = 2.8$, $\lambda_1 = 1.075$ if $\phi_1 = 30°$ and $\lambda_1 = 0.997$ if $\phi_1 = 60$ degrees.

The corresponding values for the coefficient, λ_0, are 1.67 and 1.87.* This means that if the dam is absolutely rigid against tangential displacements, the stiffness of the arch rings will be increased in a ratio of 1.075 to 1.67, or 55% for the first case and, correspondingly, 88% for the second case. Of course, the dam is not absolutely rigid against such displacements; it is only very rigid if the ratio between its span and height is relatively great. On the whole, the shear deformation in the arch does not influence the value of the coefficient, λ_1, very much. To neglect the shear deformation in the development given previously is the same as putting $\frac{\kappa E}{G}$ equal to zero, and then the coefficient, λ_1, is simplified as follows:

$$\lambda_1 = 1 - B = \frac{(\sinh w - \sinh w)(\cosh w - \cosh w)}{\sinh w \cosh w + \sin w \cos w}$$

$$= \frac{w^4}{6}\left[1 - \frac{163}{1\,260} w^4 + \ldots\right] \ldots \qquad (142)$$

in which,

$$w = a\,\phi_1 = \phi_1 \sqrt[4]{\frac{3\,r_n^{\,2}}{\rho\,t^2}} \ldots \qquad (143)$$

This formula for the coefficient, λ_1, as a function of w is demonstrated graphically in Fig. 45. If the arch is prevented from tangential displacements by means of tangential reactions (shear forces), the arch thrust at any point is proportional to the deflection as given by Equation (125). Therefore, at the abutments the thrust in such an arch is equal to zero; or, in other words, all the thrust is taken up by the tangential shear forces between the crown and the abutment.

In an actual dam this should mean that the thrust in the upper arch rings is transferred to the lower arch rings by the shear forces between the different rings. Since the structure may allow some tangential displacements, not all the arch thrust is transmitted in this way, but only a part of it. How great this is depends mainly on the ratio between span and height and the other dimensions of the dam. Imagine, for instance, that the load, p_a, taken by arch action is divided into two parts, one part, p_0, being carried by the horizontal arch thrust over to the abutments, and another part, p_1, being transferred by

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 565, Fig. 46.

tangential shear over to the other arch rings. Together these parts of the load give the total arch load:

$$p_a = p_0 + p_1 \qquad (144)$$

The first part of the load contributes to the deflection at the crown,

$$\Delta r_0 = \lambda_0 \rho \frac{p_0 r_e r_n}{E t} \qquad (145)$$

and the second part contributes,

$$\Delta r_1 = \lambda_1 \rho \frac{p_1 r_e r_n}{E t} \qquad (146)$$

or, together,

$$\Delta r = \Delta r_0 + \Delta r_1 = \lambda \rho \frac{p_a r_e r_n}{E t} \qquad (147)$$

in which,

$$\lambda = \frac{\lambda_0 p_0 + \lambda_1 p_1}{p_a} \qquad (148)$$

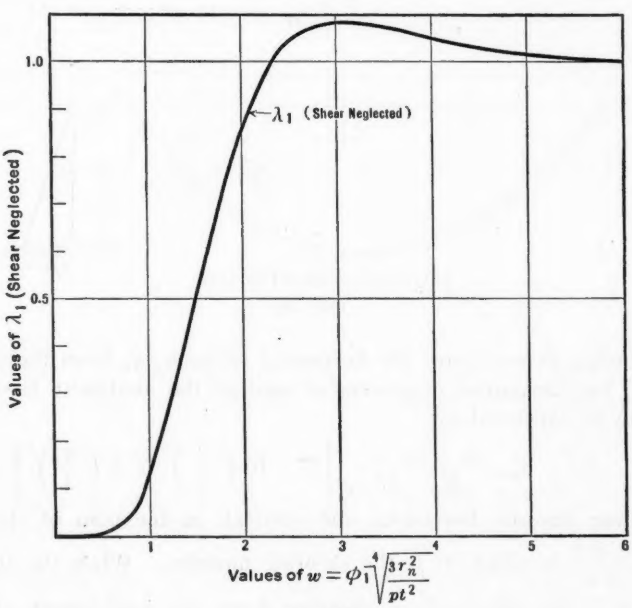

FIG. 45.

The first part of the load, p_0, gives an arch thrust carried horizontally over to the abutments as mentioned, which signifies no tangential forces between the arch rings. Instead of that, this part of the load is inseparably connected with tangential displacements, Δb, as computed by Equation (118) or Equation (119).

The other part, p_1, gives no tangential displacements according to the assumptions; but, instead, the corresponding part of the arch thrust must be taken up between the crown and the abutment by tangential shear forces, and transmitted over other arch rings vertically down to the foundation. The

total arch thrust at the center line transmitted in this way on each half of the arch is, $N = \lambda_1 \, p_1 \, r_e$, for a ring of unit thickness in a vertical direction. Of course, the shear necessary for transmitting these forces between the arch rings is just that which is producing the tangential displacements, Δb, in the cantilevers.

It is certain that this division of the arch load into two parts can only give an approximate result, since the total arch load, p_a, is not a constant along the arch and, consequently, both p_0 and p_1 cannot be constant. However, it may be useful in order to obtain an idea of the conditions.

To simplify the problem still more, consider a dam with triangular cross-section across a parabolic shaped canyon, as shown in Fig. 46. One-half the length at any elevation is,

$$l = \frac{L}{2} \sqrt{1 - \frac{x}{H}} = \phi_1 \, r_n \qquad (149)$$

and the thickness is,

$$t = \frac{T}{H} \, x \qquad (150)$$

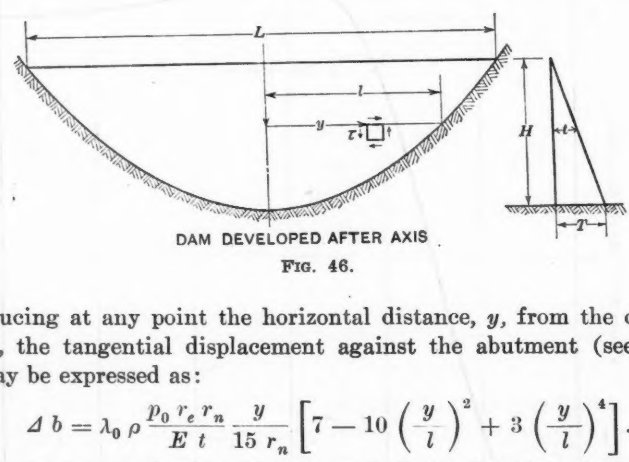

FIG. 46.

Introducing at any point the horizontal distance, y, from the center line, $y = \phi \, r_n$, the tangential displacement against the abutment (see Equation (119)) may be expressed as:

$$\Delta b = \lambda_0 \, \rho \, \frac{p_0 \, r_e \, r_n}{E \, t} \, \frac{y}{15 \, r_n} \left[7 - 10 \left(\frac{y}{l} \right)^2 + 3 \left(\frac{y}{l} \right)^4 \right] \qquad (151)$$

The shear stresses, horizontal and vertical, in the plan of the dam are $\tau = - G \dfrac{\delta \, \Delta b}{\delta \, x}$, in which, G is the shearing modulus. When the thickness of the dam is t, the difference in shearing force per unit length of the axis between the lower and upper limit of an arch ring is $\dfrac{\delta \, (\tau \, t)}{\delta \, x}$, and the total difference in this shearing force from crown to abutment must be equal to the thrust from the load, p_1:

$$\lambda_1 \, p_1 \, r_e = \int_0^l \frac{\delta \, (\tau \, t)}{\delta \, x} \, d \, y \qquad (152)$$

For the integration assume that the product, $\lambda_0 \, \rho$, is approximately constant; the variation is negligible, at least for the upper part of the arch. By substituting the different expressions given for l, t, Δb, and τ, in Equation (152), the ratio between p_1 and p_0 will be found to be equal to:

$$\frac{p_1}{p_0} = \frac{\lambda_0\,\rho}{120\,\lambda_1}\,\frac{G}{E}\,\frac{L^2}{H^2}\left[\frac{1 + 3\,\dfrac{H}{x}}{1 - \dfrac{x}{H}} - \frac{3\,H^2}{p_0}\left[\left(1 - 3\,\frac{x}{H}\right)\frac{\delta\left(\dfrac{\rho_0}{x}\right)}{\delta\,x} + x\left(1 - \frac{x}{H}\right)\frac{\delta^2\left(\dfrac{\rho_0}{x}\right)}{\delta\,x^2}\right]\right] \quad \text{(153)}$$

If Poisson's ratio, $\frac{1}{m}$, is $\frac{1}{6}$, then,

$$\frac{G}{E} = \frac{m}{2\,(m + 1)} = \frac{3}{7}$$

and $\frac{\lambda_0\,\rho}{\lambda_1}$ may be taken equal to 1.8, an average for the whole upper part of the arch.

A dam with zero thickness at the crest takes zero arch load at the crest, and both p_0 and p_1 must be zero at $x = 0$. This boundary condition is fulfilled if it is assumed that p_0 is proportional to $x\,e^{\frac{x}{H}}$, at least for small values of x. Then the ratio is,

$$\frac{p_1}{p_0} = \frac{\lambda_0\,\rho}{120\,\lambda_1}\,\frac{G}{E}\,\frac{L^2}{H^2}\,\frac{10 - 3\,\dfrac{x}{H} - 3\left(\dfrac{x}{H}\right)^2}{1 - \dfrac{x}{H}}$$

or, for $x = 0$,

$$\frac{p_1}{p_0} = \frac{\lambda_0\,\rho}{12\,\lambda_1}\,\frac{G}{E}\,\frac{L^2}{H^2} = 0.0643\,\frac{L^2}{H^2} \quad \text{(154)}$$

This function for p_0 cannot be considered valid over its entire height because that would only lead to absurd conditions. At the base all the arch load must be taken by shear since the lowest arch is directly connected with the foundation along the entire length; that is, $p_0 = 0$ and $\frac{p_1}{p_0}$ is infinitely great. The total arch load is always greater than zero at the bottom if the length profile of the dam is curved. These conditions are fulfilled, for instance, if p_0 is proportional to $x\left(1 - \frac{x}{H}\right)$, except for small values of x. It does not matter very much if p_0 is determined by a more complicated function fulfilling the conditions at the limits. Nor does it matter how the transfer between the exponential function at the top and the parabola at some section below is assumed. The main thing is only that p_0 must increase at a rate nearly proportional with x at the top and then must follow any smooth curve down to zero at $x = H$. If p_0 is found by the parabola formula, the ratio is,

$$\frac{p_1}{p_0} = \frac{\lambda_0\,\rho}{60\,\lambda_1}\,\frac{G}{E}\,\frac{L^2}{H^2}\,\frac{3\,\dfrac{H}{x} - 1}{1 - \dfrac{x}{H}} \quad \text{(155)}$$

Table 16 gives a comparison of values obtained for the ratio, $\frac{p_1}{p_0}$ by Equations (154) and (155), respectively.

TABLE 16.—COMPARATIVE VALUES OF $\frac{p_1}{p_0}$.

$\frac{x}{H}$.	p_0 proportional to $x\,e^{\frac{x}{H}}$.	p_0 proportional to $x\left(1-\frac{x}{H}\right)$.
0.	$0.064\left(\frac{L}{H}\right)^2$	....
0.1	$0.069\left(\frac{L}{H}\right)^2$	$0.414\left(\frac{L}{H}\right)^2$
0.2	$0.075\left(\frac{L}{H}\right)^2$	$0.225\left(\frac{L}{H}\right)^2$
0.3	....	$0.165\left(\frac{L}{H}\right)^2$
0.4	....	$0.139\left(\frac{L}{H}\right)^2$
0.5	....	$0.129\left(\frac{L}{H}\right)^2$
0.6	....	$0.129\left(\frac{L}{H}\right)^2$
0.7	....	$0.141\left(\frac{L}{H}\right)^2$
0.8	....	$0.177\left(\frac{L}{H}\right)^2$
0.9	....	$0.300\left(\frac{L}{H}\right)^2$
1.0	....	Infinite

As will be seen, the ratio is not a constant value, but in the central part of the dam the variation is not great. The same result will be found if other reasonable assumptions for p_0 are made. The value depends mainly on the ratio between the length, L, and the height, H.

A similar analysis can be easily made for other cross-sections and canyon profiles. Some difficulties may result if the arches near the crest are designed too strong, because this may sometimes lead to a negative value for the part, p_1, of the load. However, the fact remains that a certain fraction of the arch load, the fraction depending mainly on the ratio, $\frac{L^2}{H}$, must be transmitted vertically down to the foundation by tangential shear between the arch rings. This fraction is found to be smaller than in the case of a triangular section if the thickness of the dam is the same for different elevations, as in the Stevenson Creek Dam. Since practical dams always have a more or less triangular-shaped cross-section, the writer has used the following formula in the computation of stresses in some dams:

$$R = \frac{p_1}{p_0} = \frac{1}{10}\left(\frac{L}{H}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots(156)$$

It is possible that,

$$R = \frac{p_1}{p_0} = \frac{1}{8}\left(\frac{L}{H}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots(157)$$

would give a better average.

The stiffness against tangential shear deformation is taken into account in this way, at least approximately, and without any noticeable increase in the numerical calculation. Equation (148) is then found to be:

$$\lambda = \frac{\lambda_0 + R\,\lambda_1}{1 + R} \qquad (158)$$

Both λ_0 and λ_1 can be taken directly from the diagrams, and R is the ratio given by Equation (156) or Equation (157).

For instance, for the Gibson Dam, the length is about 4.5 times the height, the ratio between the coefficients, λ_0 and λ_1, is about 1.8 for the greatest part of the dam, and the stiffness of the arch rings against deflection in this way is found to be increased by about 40 to 45%; that is, with the same deflection, the arch load is increased by 40 to 45% due to this tangential stiffness. That, of course, will also change the load distribution between the arches and the cantilevers. However, the change in load will be less than 40% since the deflection also will be changed.

For arch dams in narrow canyons, the influence will be much less. The tangential stiffness is only of importance at wide dam sites, in which case it is a factor that cannot be neglected in any calculation of stresses. The next question is how to take it into account (at least, approximately) in a way that will not cause too much additional work.

Where only the deflection of one cantilever at the center is compared with the deflection of the arch rings, and the load on each arch ring is assumed to be uniformly distributed, the use of the coefficient, λ_1 (Equation (158)), instead of λ_0, at once gives the solution. As far as actual arch dams with approximately triangular cross-section and large central angles constructed in U-shaped canyons are concerned, the writer has found that these very simplified assumptions give the bending stresses in the highest cantilevers within a few per cent., and even give the arch stresses without very great errors. The distribution of load for that case may be found in one to two days instead of one to two months by the complete trial-load method.

Where it is found necessary to take the deflection of more than the central cantilever into account in the computations, it may be convenient to use the following procedure: First, compute the approximate division of load by taking only the central cantilever into account as before, and by the arch-load diagram and the approximate division of the arch load in the two components, p_0 and p_1, calculate the approximate values of the horizontal arch thrust and the tangential shear force for a number of points at different vertical planes of the dam. By these values for arch thrust and shear the direction of the true thrust can be found in the same way as the principal stresses in an infinitely small cube. The slope of this direction may be ψ. Then,

$$\tan 2\,\psi = 2\,\frac{\text{shear per unit length of arch}}{\text{thrust per unit height} - \text{vertical force per unit length of arch}}$$

After the declination of the thrust due to water pressure is found, at least approximately, the trajectories of thrust are plotted and the dam is divided into a number of arch rings limited by these trajectories as shown in Fig. 47. Between these arch rings no shear of any importance can occur, and they may be used for analysis instead of the horizontal arch rings used by the authors.

The vertical component of the inclined arch thrust transfers part of the weight of the cantilevers at the center to the abutments of the different arch rings. These vertical forces can be computed directly from the horizontal forces, since the resultant at any point must follow the trajectories. The complication in the numerical computations then is mainly reduced to the use of arch rings of variable thickness, both vertically and horizontally, instead of rings with only the horizontal thickness being variable. The yielding of the foundation may be taken into account in the same manner as for the other arch rings.*

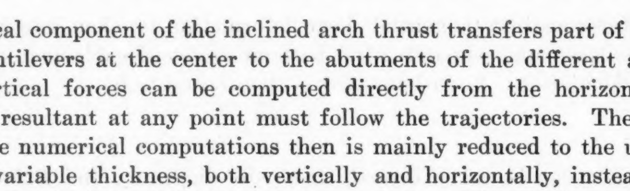

FIG. 47.

The results of the analysis will be changed as follows:

1.—The load taken by the arches is increased and the load taken by the cantilevers reduced, which reduces the bending stresses in the cantilevers.

2.—Part of the weight of the dam at the center will be carried by the vertical components of the arch thrust, and transmitted to either side of the dam. This reduces the compression stresses and increases possible tensile stresses at the central cantilevers and *vice versa* at the cantilevers near the abutments.

3.—The actual arch rings are much stiffer at the abutments than the horizontal arch rings, since the thickness there is increased both vertically and horizontally from the thickness at the crown, which may give quite a different distribution of the arch stresses.

Some of the dams referred to in the paper are made considerably thicker at the springing line than at the crown. It seems doubtful whether an extended analysis of these problems would justify this detail since the difference already is obtained with constant thickness in horizontal planes by using the true inclined arch rings.

A. FLORIS,† ESQ. (by letter).‡—The analysis of arches and particularly of arch dams has been treated repeatedly in recent years. Various writers have proposed formulas for the purpose of improving those in current use. This paper is another attempt in this direction in spite of the fact that the theory of arches has already been developed fairly well.

Equations (27),§ (28), and (29),‖ also found elsewhere,¶ are not "perfectly general" because they neglect the work done by the shearing forces which,

* See discussion by Messrs. Savage and Houk, *Proceedings*, Am. Soc. C. E., October. 1928, Papers and Discussions, p. 2317.

† Los Angeles, Calif.

‡ Received by the Secretary, August 8, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 83.

‖ *Loc. cit.*, p. 84.

¶ "Plain and Reinforced Concrete Arches," by J. Melan; translated by D. B. Steinman, M. Am. Soc. C. E., N. Y., 1915, p. 30.

in dams, is of considerable importance. They also omit the influence of a difference in temperature between the extrados and intrados of the arch.

The writer wishes to present some more general expressions for the statically indeterminate quantities which include these influences. Furthermore, it will be shown that a close relation exists between these general equations and the various well-known formulas. By choosing a suitable origin of co-ordinates to which the fundamental equations are referred and by successive simplifications, these special formulas can be easily derived.

It is well known that a curved prismatic bar restrained at its supports is a statically indeterminate system in the third degree. For these unknown quantities, therefore, three equations of deformation are necessary. There are a number of methods of analysis available which provide these equations. The theorem of the derivatives of the internal work will be used herein because it leads to the solution of the problem in a most simple manner.

Generally, the theory of structures is based on assumptions* in order to make a practical treatment of the subject possible. With this object in view the writer has made the usual assumptions that Hooke's law and Bernoulli's hypothesis that plane sections remain plane after bending, are valid and that the shearing stresses are distributed according to a parabolic law.

If the radius of curvature of a curved bar is not less than about five times its thickness† the general equation of the internal work with all possible effects included is:‡

$$u = \int \frac{M^2\, d\, s}{2\, E\, I} + \int \frac{N^2\, d\, s}{2\, E\, A} + \kappa \int \frac{Q^2\, d\, s}{2\, G\, A} + \int \varepsilon \frac{\varDelta\, t}{h}\, M\, d\, s + \int \varepsilon\, t\, N\, d\, s \ldots (159)$$

Differentiating Equation (159) with respect to X, Castigliano's theorem of partial derivatives is obtained:

$$L = \frac{\delta\, u}{\delta\, X} = \int \frac{M}{E\, I} \frac{\delta\, M}{\delta\, X} d\, s + \int \frac{N}{E\, A} \frac{\delta\, N}{\delta\, X} d\, s + \kappa \int \frac{Q}{G\, A} \frac{\delta\, Q}{\delta\, X} d\, s$$
$$+ \int \varepsilon \frac{\varDelta\, t}{h} \frac{\delta\, M}{\delta\, X} d\, s + \int \varepsilon\, t \frac{\delta\, N}{\delta\, X} d\, s \ldots\ldots\ldots\ldots\ldots (160)$$

If the supports are rigid, this expression will be:

$$\int \frac{M}{E\, I} \frac{\delta\, M}{\delta\, X} d\, s + \int \frac{N}{E\, A} \frac{\delta\, N}{\delta\, X} d\, s + \kappa \int \frac{Q}{G\, A} \frac{\delta\, Q}{\delta\, X} d\, s + \int \varepsilon \frac{\varDelta\, t}{h} \frac{\delta\, M}{\delta\, X} d\, s$$
$$+ \int \varepsilon\, t \frac{\delta\, N}{\delta\, X} d\, s = 0 \ldots\ldots\ldots\ldots\ldots\ldots (161)$$

which is the theorem of least work.

By using these equations, any prismatic bar can be analyzed in the shortest and most elegant manner. On this fact is based the superiority of the theorem of internal work as compared with the cumbersome and lengthy method of consistent deformations preferred by the authors.

* "Calcul des arcs élastiques," par S. Timoshenko, Paris, 1922, pp. 4–6.

† "Applied Elasticity," by S. Timoshenko and J. M. Lessels, East Pittsburgh, 1925, p. 221; or "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," von H. Mueller-Breslau, Berlin, 1924, p. 242.

‡ "Kinetic Theory of Engineering Structures," by D. A. Molitor, M. Am. Soc. C. E., N. Y., 1911, p. 42.

The integrals in Equations (159), (160), and (161) are those due to bending moments, normal forces, shearing forces, unequal temperature variation within the section of the bar, and uniform temperature change throughout the section, respectively.

In these equations, $G = E \dfrac{m}{2\,(m+1)}$*, $\nu = 2\,\kappa \dfrac{m+1}{m}$, and for rectangular sections, $\kappa = \dfrac{6}{5}$†.

Substituting these values, the integral of the shearing forces in Equations (159), (160), and (161) becomes,

$$\nu \int \frac{Q}{E\,A}\,\frac{\delta\,Q}{\delta\,x}\,d\,s$$

Concrete tests show‡ that m varies between 6.5 and 8.0. Taking an average value of 7.25, then $\nu = 2.74$ and not 3.0 or 2.88 as is assumed by various writers.

A restrained arch which is either unsymmetrically formed or unsymmetrically loaded is statically indeterminate in the third degree. Taking any point as the origin of the co-ordinates and introducing X, Y, and μ as the statically indeterminate quantities, the bending moment, normal force, and shearing force for any point on the arch axis will be (see Fig. 56):

$$M = M_0 + X\,y + Y\,x + \mu \quad\quad (162)$$

$$N = N_0 + X \cos\phi + Y \sin\phi \quad\quad (163)$$

$$Q = Q_0 - X \sin\phi + Y \cos\phi \quad\quad (164)$$

The corresponding derivatives will be:

$$\frac{\delta\,M}{\delta\,X} = y; \quad \frac{\delta\,M}{\delta\,Y} = x; \quad \frac{\delta\,M}{\delta\,\mu} = 1 \quad\quad (165)$$

$$\frac{\delta\,N}{\delta\,X} = \cos\phi; \quad \frac{\delta\,N}{\delta\,Y} = \sin\phi; \quad \frac{\delta\,N}{\delta\,\mu} = 0 \quad\quad (166)$$

$$\frac{\delta\,Q}{\delta\,X} = -\sin\phi; \quad \frac{\delta\,Q}{\delta\,Y} = \cos\phi; \quad \frac{\delta\,Q}{\delta\,\mu} = 0 \quad\quad (167)$$

M_0, N_0, and Q_0, in Equations (162) to (164) are the bending moment, normal force, and shearing force, respectively, of the statically determinate system which is sometimes called the principal or base system.

Because there are three unknowns, X, Y, and μ, Equation (161) must be written out three times as follows:

$$\int_A^B \frac{M}{E\,I}\,\frac{\delta\,M}{\delta\,X}\,d\,s + \int_A^B \frac{N}{E\,A}\,\frac{\delta\,N}{\delta\,X}\,d\,s$$
$$\int_A^B \frac{Q}{E\,A}\,\frac{\delta\,Q}{\delta\,X}\,d s + \int_A^B \varepsilon\,\frac{\Delta\,t}{h}\,\frac{\delta\,M}{\delta\,X}\,d\,s + \int_A^B \varepsilon\,t\,\frac{\delta\,N}{\delta\,X}\,d\,s = 0. \quad (168)$$

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* "Strength of Materials," by G. F. Swain, Past-President, Am. Soc. C. E., N. Y., 1924, p. 62.

† "Théorie de l'équilibre des systèmes élastiques et ses applications," Turin, 1879. English translation by E. S. Andrews, Lond., 1919, p. 226.

‡ "Researches in Concrete," by W. K. Hatt, M. Am. Soc. C. E., *Bulletin No. 24*, Eng. Experiment Station, Purdue Univ., p. 38.

$$\int_A^B \frac{M}{E\,I}\frac{\delta M}{\delta Y} d\,s + \int_A^B \frac{N}{E\,A}\frac{\delta N}{\delta Y} d\,s + \nu \int_A^B \frac{Q}{E\,A}\frac{\delta Q}{\delta Y} d\,s$$
$$+ \int_A^B \varepsilon \frac{\Delta\,t}{h}\frac{\delta M}{\delta Y} d\,s + \int_A^B \varepsilon\,t \frac{\delta N}{\delta Y} d\,s = 0 \ldots\ldots\ldots (169)$$

$$\int_A^B \frac{M}{E\,I}\frac{\delta M}{\delta \mu} d\,s + \int_A^B \frac{N}{E\,A}\frac{\delta N}{\delta \mu} d\,s + \nu \int_A^B \frac{Q}{E\,A}\frac{\delta Q}{\delta \mu} d\,s$$
$$+ \int_A^B \varepsilon \frac{\Delta\,t}{h}\frac{\delta N}{\delta \mu} d\,s + \int_A^B \varepsilon\,t \frac{\delta N}{\delta \mu} d\,s = 0 \ldots\ldots\ldots (170)$$

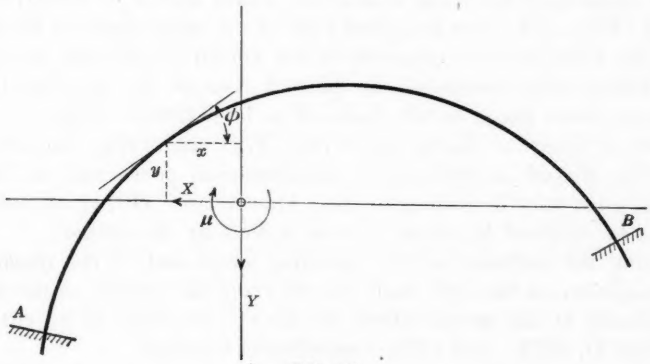

FIG. 56.

In Equations (168), (169), and (170), it is assumed that the uniform temperature variation in the arch is acting in the same direction with the outer forces. This is the case when a drop in temperature takes place in the arch. For a rise in temperature the sign before the integral, expressing this uniform change, must be reversed. Introducing the values given by Equations (162) to (167) in the proper place in Equations (168), (169), and (170), the following expressions are derived:

$$X\left[\int_A^B \frac{y^2}{I} d\,s + \int_A^B \frac{\cos^2\phi}{I} d\,s + \nu \int_A^B \frac{\sin^2\phi}{I} d\,s\right]$$
$$+ Y\left[\int_A^B \frac{x\,y}{I} d\,s + (1-\nu)\int_A^B \frac{\sin\phi\cos\phi}{I} d\,s\right]$$
$$+ \mu \int_A^B \frac{y}{I} d\,s + \int_A^B \frac{M_0}{I} y\,d\,s + \int_A^B \frac{N_0}{A}\cos\phi\,d\,s - \nu \int_A^B \frac{Q_0}{A}\sin\phi\,d\,s$$
$$+ E\,\varepsilon \int_A^B \frac{\Delta\,t}{h} y\,d\,s + E\,\varepsilon\,t \int_A^B \cos\phi\,d\,s = 0 \ldots\ldots\ldots (171)$$

$$X\left[\int_A^B \frac{y\,x}{I} d\,s + (1-\nu)\int_A^B \frac{\sin\phi\cos\phi}{A} d\,s\right] + Y\left[\int_A^B \frac{x^2}{I} d\,s\right.$$
$$\left. + \int_A^B \frac{\sin^2\phi}{A} d\,s + \nu \int_A^B \frac{\cos^2\phi}{A} d\,s\right]$$
$$+ \mu \int_A^B \frac{x}{I} d\,s + \int_A^B \frac{M_0}{I} x\,d\,s + \int_A^B \frac{N_0}{A}\sin\phi\,d\,s + \nu \int_A^B \frac{Q_0}{A}\cos\phi\,d\,s$$
$$+ E\,\varepsilon \int_A^B \frac{\Delta\,t}{h} x\,d\,s + E\,\varepsilon\,t \int_A^B \sin\phi\,d\,s = 0 \ldots\ldots\ldots (172)$$

$$X\int_A^B \frac{y}{I}\,d\,s + Y\int_A^B \frac{x}{I}\,d\,s + \mu\int_A^B \frac{d\,s}{I} + \int_A^B \frac{M_0}{I}\,d\,s + E\,\varepsilon\int_A^B \frac{\Delta\,t}{h}\,d\,s = 0 \quad (173)$$

Equations (171), (172), and (173) are the most general expressions for the statically indeterminate quantities of an unsymmetrically formed and unsymmetrically loaded arch with complete restraint at its abutments. However, if the abutments are not perfectly rigid then, instead of being equal to zero, Equations (168), (169), and (170) must be made equal to these elastic movements; or, what is the same thing, Equation (160) must be used. Obviously, the expressions for these movements would appear in Equations (171), (172), and (173), which can be solved only if the magnitudes of the displacements or the rotations are measured or are known in advance in some way. For this reason these formulas are omitted because, in practice, the abutments of arch dams are generally assumed to be perfectly rigid.

In order to integrate Equations (171), (172), and (173), the axis of the arch must be shaped according to a mathematical curve and the thickness of the arch must vary according to some known law. Otherwise, the desired result must be obtained by summation as is done by the authors.

Neglecting the influence of the shearing forces and of the unequal temperature variation on the arch and also referring the system to the origin of the co-ordinates at the crown where the arch is assumed to be cut in two, Equations (171), (172), and (173) immediately become:

$$X\left[\int_A^B \frac{y^2}{I}\,d\,s + \int_A^B \frac{\cos^2\phi}{A}\,d\,s\right] + Y\left[\int_A^B \frac{x\,y}{I}\,d\,s + \int_A^B \frac{\sin\phi\cos\phi}{A}\,d\,s\right]$$
$$+\,\mu\int_A^B \frac{y}{I}\,d\,s + \int_A^B \frac{M_0}{I}\,y\,d\,s + \int_A^B \frac{N_0}{A}\cos\phi\,d\,s + E\,\varepsilon\,t\int_A^B \cos\phi\,d\,s = 0 \quad (174)$$

$$X\left[\int_A^B \frac{y\,x}{I}\,d\,s + \int_A^B \frac{\cos\phi\sin\phi}{A}\,d\,s\right] + Y\left[\int_A^B \frac{x^2}{I}\,d\,s + \int_A^B \frac{\sin^2\phi}{A}\,d\,s\right]$$
$$+\,\mu\int_A^B \frac{x}{I}\,d\,s + \int_A^B \frac{M_0}{I}\,x\,d\,s + \int_A^B \frac{N_0}{A}\sin\phi\,d\,s + E\,\varepsilon\,t\int_A^B \sin\phi\,d\,s = 0 \quad (175)$$

$$X\int_A^B \frac{y}{I}\,d\,s + Y\int_A^B \frac{y}{I}\,d\,s + \mu\int_A^B \frac{d\,s}{I} + \int_A^B \frac{M_0}{I}\,d\,s = 0 \quad (176)$$

which are the authors' Equations (29), (28), and (27), in a somewhat different form.

In these expressions the integration must be extended over the entire arch. However, bearing in mind that in Equations (162) and (163) the terms containing Y must be negative for the right half of the arch, the integrals in Equation (175) are obtained by subtracting the integrals of the right half from those of the left half. On the other hand, x^2 and $\sin^2\phi$ being positive, the corresponding integrals must be added and not subtracted. In the same way the integrals of Y in Equations (174) and (176) must be also subtracted. The remaining integrals are the sums of the left and right-hand parts of the arch.

Considering, furthermore, that the integrals, $\int_A^B \cos\phi\,d\,s$ and $\int_A^B \sin\phi\,d\,s$, are the projections of the arch upon the x and y-axes, respectively, it follows

therefrom that they define the distances between the projections on the x and y-axes of the points, A and B, of the arch at its abutments, respectively. With this fact in view the term denoting the influence of the uniform temperature variation in the author's Equation (29) seems to be incorrect.

In a symmetrical arch, but with an unsymmetrical loading, the integrals corresponding to the left and right-hand parts of the arch are equal, so that their difference will be zero.

In this case, therefore, the effect will be to make the integrals of Y in Equations (174) and (176) and the integrals of X and μ in Equation (175) equal to zero. At the same time the last integral in Equation (175) vanishes also for the reason that the deformation of the arch due to a uniform temperature variation being symmetrical no Y-force can be produced. This result is based on the supposition that the arch is cut in two at the crown.

This choice of the principal system facilitates the computations appreciably because, by taking the origin of the co-ordinates at the left abutment, for instance, and cutting the arch at this point, Equations (174), (175), and (176) must be used even in a symmetrical arch under unsymmetrical loading. It is needless to say that in this case the integrals in these equations are the sums and not the differences of the two halves of the arch.

In bridges the thickness of the arch is small as compared with the radius and, consequently, the work done by the shearing forces is always neglected. The deformation due to the normal forces is also neglected if the arch is not too flat. In Equations (168), (169), and (170), therefore, the integrals due to the bending moments alone are taken into consideration and the general Equations (171), (172), and (173), neglecting the influence of temperature, are simplified as follows: In Equation (171), the second, third, fifth, eighth, ninth, tenth, and eleventh integrals vanish; in Equation (172), the second, fourth, fifth, eighth, ninth, tenth, and eleventh integrals are equal to zero; and in Equation (173), the last integral disappears.

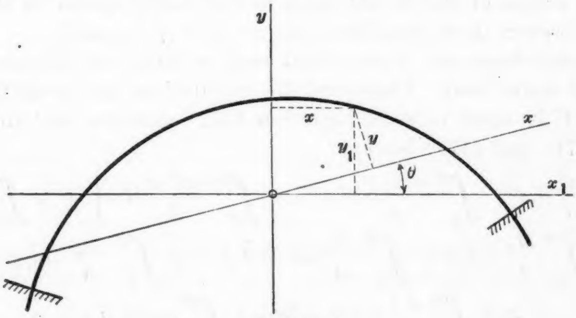

FIG. 57.

Müller-Breslau was the first to show* that the integrals, $\int_A^B \frac{y}{I}\,d\,s$ and $\int_A^B \frac{x}{I}\,d\,s$, representing the static moments and $\int_A^B \frac{x\,y}{I}\,d\,s$, the centrifugal moment of the

* "Vereinfachung der Theorie der statisch unbestimmten Bogentraeger," H. von Müller-Breslau, *Zeitschrift* des Architekten und Ingenieur Vereins zu Hannover, 1884, p. 575.

symmetrical arch ring, vanish if they are referred to the elastic center of gravity of the ring. The elastic center, on the other hand, is determined by considering $\frac{d\,s}{I}$ as forces acting in two directions. In the case of a symmetrical arch the y-axis being an axis of symmetry the condition, $\int_A^B \frac{x}{I}\, d\,s = 0$, is satisfied. However, if the arch is unsymmetrical, the centrifugal moment, $\int_A^B \frac{x\,y}{I}\, d\,s$, vanishes only in the case when it is referred to an inclined axis, the slope of which is given by* (see Fig. 57):

$$t\,g\,\theta = \frac{\int_A^B \frac{x\,y_1}{I}\, d\,s}{\int_A^B \frac{x^2}{I}\, d\,s}$$

The great advantage of this arrangement is that all the more complicated formulas are reduced to three single equations with one unknown each. They are obtained as follows: In Equation (171) all integrals except the first and seventh are equal to zero; in Equation (172) all integrals vanish with the exception of the third and seventh; and, lastly, in Equation (173) all integrals, except the third and fourth, are equal to zero.

In arch dams, in which the work done by the normal forces usually predominates, there are no methods available to reduce Equations (171), (172), and (173) to three equations with one unknown each, although sometimes this is erroneously thought possible.† It is true that in bridges with a comparatively small rise it can be assumed approximately in Equation (163) that $X \cos \phi = X$ and $Y \sin \phi = 0$, if a special analysis is made for the dead load of the arch.‡ With the aid of these assumptions it is possible to reduce Equations (171), (172), and (173), to three equations with one unknown each by referring the origin of the co-ordinates to the elastic center of the arch. In arch dams, however, these simplifications are not permissible.

Usually, arch dams are symmetrical and, as such, are always stressed by a symmetrical water load. Therefore, the calculations are simplified considerably. Since Y is equal to zero, Equation (172) vanishes and the remaining Equations (171) and (173) become:

$$X\left[\int_A^B \frac{y^2}{I}\, d\,s + \int_A^B \frac{\cos^2\phi}{A}\, d\,s + \nu \int_A^B \frac{\sin^2\phi}{A}\, d\,s\right] + \mu \int_A^B \frac{y}{I}\, d\,s$$
$$+ \int_A^B \frac{M_0}{I}\, y\, d\,s + \int_A^B \frac{N_0}{A} \cos\phi\, d\,s - \nu \int_A^B \frac{Q_0}{A} \sin\phi\, d\,s$$
$$+ E\,\varepsilon \int_A^B \frac{\Delta t}{h}\, y\, d\,s + E\,\varepsilon\, t \int_A^B \cos\phi\, d\,s = 0 \quad \ldots\ldots\ldots (177)$$

$$X \int_A^B \frac{y}{I}\, d\,s + \mu \int_A^B \frac{d\,s}{I} + \int_A^B \frac{M_0}{I}\, d\,s + E\,\varepsilon \int_A^B \frac{\Delta t}{h}\, d\,s = 0 \quad ..(178)$$

* "Gewoelbte Bruecken," von E. Moersch, *Beton-Kalender*, Berlin, 1915, p. 303.

† "Arch Dams with Rings of Variable Thickness", by F. W. Hanna and T. L. E. Haug, Members, Am. Soc. C. E., *Western Construction News*, May 25, 1928.

‡ "Gewoelbte Bruecken," von E. Moersch, *Beton-Kalender*, Berlin, 1915, pp. 295 and 302.

These are the fundamental equations derived by Professor Cain arranged in a more general form.* They are referred to the origin of the co-ordinates at the crown.

Assuming the elastic center of the arch as the origin of the co-ordinates, then, $\int_A^B \frac{y}{I}\, d\, s = 0$, as stated previously, and Equations (177) and (178) become:

$$X\left[\int_A^B \frac{y^2}{I}\, d\, s + \int_A^B \frac{\cos^2 \phi}{A}\, d\, s + \nu \int_A^B \frac{\sin^2 \phi}{A}\, d\, s\right] + \int_A^B \frac{M_0}{I}\, y\, d\, s$$
$$+ \int_A^B \frac{N_0}{A} \cos \phi\, d\, s - \nu \int_A^B \frac{Q_0}{A} \sin \phi\, d\, s + E\, \varepsilon \int_A^B \frac{\Delta\, t}{h}\, y\, d\, s$$
$$+ E\, \varepsilon\, t \int_A^B \cos \phi\, d\, s = 0 \qquad (179)$$

$$\mu \int_A^B \frac{d\, s}{I} + \int_A^B \frac{M_0}{I}\, d\, s + E\, \varepsilon \int_A^B \frac{\Delta\, t}{h}\, d\, s = 0 \qquad (180)$$

in which the unknowns, X and μ, are separated. Similar expressions, in a less complete form, were developed by Dr. N. Kelen for arches of constant thickness.‡

When it is required to determine the influence of an unequal temperature variation on an arch of constant thickness, h, Equation (179) vanishes entirely because,

$$E\, \varepsilon \frac{\Delta\, t}{h} \int_A^B y\, d\, s = 0$$

in which,

$$\int_A^B \frac{y}{I}\, d\, s = \int_A^B y\, d\, s = 0$$

since it is referred to the elastic center of the arch. The remaining Equation (180), therefore, will be,

$$\mu \int_A^B \frac{d\, s}{I} + E\, \varepsilon \frac{\Delta\, t}{h} \int_A^B d\, s = 0 \qquad (181)$$

Substituting for I its equivalent, $\frac{h^3}{12}$:

$$\mu = E\, \varepsilon \frac{h^2}{12} \Delta\, t‡ \qquad (182)$$

in which, $\Delta\, t$ is the difference, $t_1 - t_2$, between the up-stream and down-stream temperatures. This difference determines the sign of μ since compression is developed in the warmer side of the arch.

One can introduce a further simplification by designing the arch so that the line of resistance in the principal system coincides exactly with the arch axis. In this case $\int_A^B \frac{M_0}{I}\, y\, d\, s$ and $\int_A^B \frac{M_0}{I}\, d\, s$ in Equations (179) and (180) vanish; consequently, $\int_A^B \frac{Q_0}{I} \sin \phi\, d\, s$ is equal to zero, since the shearing forces are produced by the difference of the moments in two adjacent sections of the

* "The Circular Arch Under Normal Loads," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Equations (7) and (9), p. 237.

† "Die Staumauern," von N. Kelen, Berlin, 1926, p. 17.

‡ "Statica delle dighe per laghi artificiali," C. Guidi, Turin, 1921, p. 49.

arch which presumably are equal to zero. Equations (179) and (180), therefore, take the extremely simple form:

$$X\left[\int_A^B \frac{y_2}{I}\,d\,s+\int_A^B \frac{\cos^2\phi}{A}\,d\,s+\nu\int_A^B \frac{\sin^2\phi}{A}\,d\,s\right]$$

$$+\int_A^B \frac{N_0}{A}\cos\phi\,d\,s+E\,\varepsilon\int_A^B \frac{\Delta\,t}{h}\,y\,d\,s+E\,\varepsilon\,t\int_A^B \cos\phi\,d\,s=0\ldots.(183)$$

which is Professor E. Moersch's generalized equation.*

It is unfortunate that this elegant and simple method of analysis (Equation (183)) has found little favor among American designers although it was made known several years ago.†

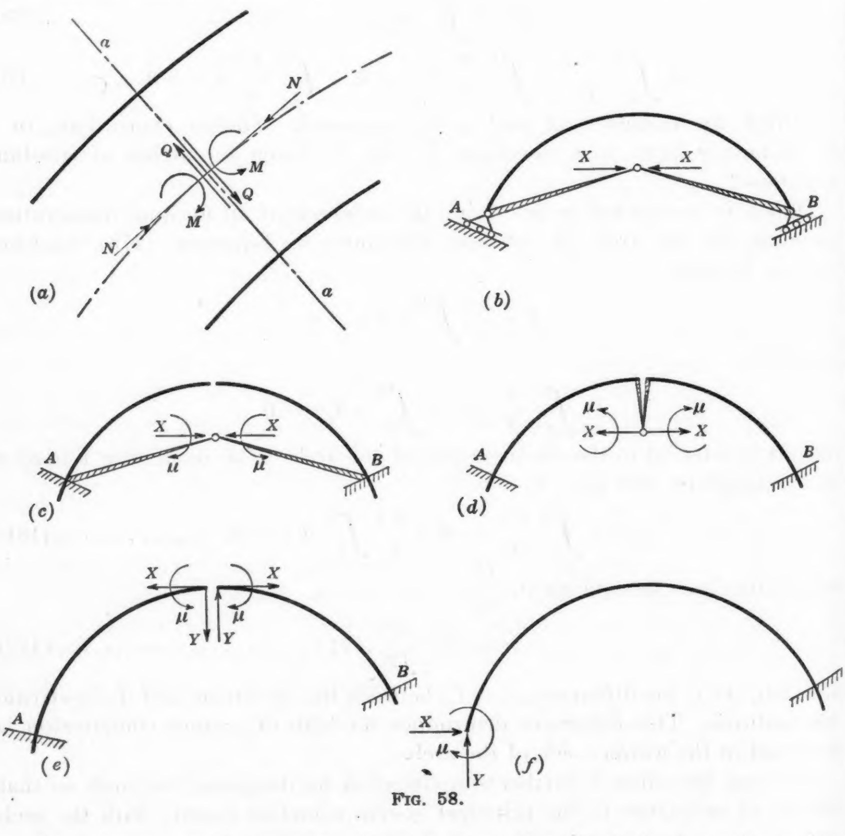

FIG. 58.

In order to apply correctly the equations already derived to the various principal systems it must be remembered that the positive directions of N, Q, and M to the left and to the right of a given section, *a-a*, are as shown in Fig. 58(*a*). It is also assumed that a positive bending moment, M, produces compression in the extrados of the arch.

* "Berechnung Kreisfoermiger Gewoelbe gegen Wasserdruck," von E. Moersch, *Schweizerische Bauzeitung*, 1908, p. 233.

† "The Relation Between Deflections and Stresses in Arch Dams," by F. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 302.

With these definitions in mind the statically indeterminate quantities, X, Y, and μ, in Fig. 58(*b*), 58(*c*), and 58(*f*), being reactions, are acting to the left of a given section, *a-a*. On the other hand, the same quantities in Fig. 58(*d*) and 58(*e*) belong to the arch itself and, consequently, they must act to the left side of this section.

In both cases, Fig. 58(*c*) and (*d*), the moment due to X is, for sections below the x-axis, positive, and, for sections above this axis, negative.

The bending moment, M_0, of the curved cantilever, Fig. 58(*c*), (*d*), (*e*), and (*f*), is negative because it produces tension in the extrados of the arch. On the other hand, the principal system, Fig. 58(*b*), is free of bending moments, M_0.

The force, X, acting in the elastic center of the arch is sometimes called a pool,* which is misleading from the point of view of precise thinking. This force is a compression, except in the case of a rise in temperature in the arch, and its relative direction depends only on the choice of the principal system.

As regards the most suitable principal system for a given case the following might be of interest.

There are many methods of obtaining this principal system, depending entirely on the ease of making the computations. In a symmetrical arch, the principal system represented in Fig. 58(*b*) is convenient for uniformly distributed water load and also uniform change of temperature or shrinkage, while either of the principal systems Fig. 58(*c*) and (*d*) are preferable for the dead load of the arch or for a variable water pressure.† In other words, the first case is applicable to the arch dams while the second case is to be used in multiple-arch dams. The unequal temperature variation between the extrados and intrados of the arch belongs to either of the cases shown in Fig. 58(*b*), (*c*), or (*d*).

If the arch is circular and of constant thickness and the water pressure is uniform, the line of resistance in the principal system coincides with its geometrical axis. In this case Equation (183) is strictly correct. For a variable water pressure and for the dead load of an arbitrarily formed arch, the line of resistance does not coincide with its axis and, consequently, Equations (179) and (180) must be used. However, Equation (183) can be applied also with a sufficient degree of accuracy if the condition that $M_0 = 0$ is not strictly satisfied.‡

In an unsymmetrical arch under water pressure there are no advantages in choosing the elastic center of the arch as the origin of the co-ordinates. For this reason it can be assumed that this origin is either at the crown or at the left abutment. In the first case the principal system consists of two cantilevers, Fig. 58(*e*), while in the second case one cantilever stretches over the entire span of the arch, Fig. 58(*f*). To both cases Equations (174), (175), and (176) must be applied.

* "The Design and Construction of Dams," by E. Wegmann, M. Am. Soc. C. E., N. Y., 1927, p. 445.

† "Die Staumauern," von N. Kelen, p. 20.

‡ "Improved Type of Multiple-Arch Dam," by F. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 361.

With these definitions in mind the statically indeterminate quantities, [illegible] and a in Fig. 50(b), 50(c), and 50(d), being functions, are acting to the [illegible] of a given section x_0. On the other hand the same quantities in Fig. [illegible] and [illegible] belong to the system itself and, consequently, they must not be [illegible] left side of the section.

In both cases, Fig. 50(c) and (d), the moment due to [illegible] for sections [illegible] the arch, is [illegible] and, for sections above the axis, negative.

The bending moment, M_0, of the curved cantilever, Fig. 50(b), (c), [illegible] is negative because it produces tension in the extrados of the arch. On the other hand, the principal system, Fig. 50(d), is free of bending moments M_0.

The force X acting in the elastic center of the arch is sometimes called [illegible] which is misleading, from the point of view of present thinking. [illegible] quantity, its direction, except in the case of an arch symmetrical in the [illegible] and its counter direction depends only on the choice of the principal [illegible].

[illegible] regards the most suitable principal system for a given case the following [illegible] of interest.

There are many methods of obtaining the principal system depending [illegible] on the ease of making the computations. In a symmetrical arch, the principal system represented in Fig. 50(d) is convenient for uniformly distributed water load and also uniform change of temperature or shrinkage, while either of the principal systems Fig. 50(b) and (c) are preferable for the [illegible] of the analysis for a variable water pressure.§ In other words the first case is applicable to the arch dams while the second case is to be used for multiple-arch dams. The unequal temperature variation between the [illegible] and intrados of the arch necessitates either of the cases shown in Fig. [illegible] or (c).

If the arch is circular and of constant thickness and the water pressure is uniform the line of resistance in the principal system coincides with its [illegible] axis. In this case Equation (187) is strictly correct. For a [illegible] water pressure and for the dead load of an arbitrarily formed arch, the line of resistance does not coincide with its axis and, consequently, Equations (180) and (190) lose the value. However, Equation (187) can be applied [illegible] with a sufficient degree of accuracy if the condition that $M_0 = 0$ is not strictly satisfied.‡

[illegible] an unsymmetrical arch under water pressure there are no advantages [illegible] taking the elastic center of the arch as the origin of the co-ordinates. For this reason it can be assumed that the origin is either at the crown or at the left abutment. In the first case the principal system consists of two cantilevers, Fig. 50(e), while in the second one one cantilever stretches over the entire span of the arch, Fig. 50(f). In both cases Equations (171), (172), and (173) must be applied.

[illegible] Design and Construction of Dams" by E. Wegmann, [illegible]

[illegible] Statik der [illegible]

[illegible] "Design of [illegible] Arch Dams," by [illegible] Trans. Am. Soc. C. E., Vol. LXXXVI [illegible]

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE COMPRESSIBILITY OF SAND-MICA MIXTURES

Discussion*

By D. P. Krynine, M. Am. Soc. C. E.

D. P. Krynine,† M. Am. Soc. C. E. (by letter).‡—The author is entirely correct when he states§ that:

"* * * the study of foundation problems by investigation of the physics and mechanics of soils is the only method which may be expected to yield results of present utility and of ultimate value to the Engineering Profession."

The writer accepts the word, "foundations", in its broad sense as meaning not only building foundations, but also highway foundations and sand-clay roads, which are foundations for supporting vehicles.

Under the caption, "Methods of Investigations",‖ some modifications are suggested. There is no analogy between soil and other engineering materials. Steel, wood, etc., are more or less homogeneous masses of atoms and molecules, and soil is a system of solids, each member of which can have a movement of its own. Therefore, if in the investigations of steel, wood, etc., statical methods predominate, in clay investigations dynamical processes must not be ignored. The author recognizes this implicitly by indicating¶ that the apparatus developed by Professor Terzaghi furnishes data as to the voids in clays referred to both pressure and time. The three co-ordinates, x, y, and z, thus became insufficient for characterizing the clay deformations; and the fourth co-ordinate, t, was introduced, which is just the attribute of dynamics. Furthermore, in any other engineering material there is no such intimacy between the material and the water as there is in clay. Water which fills

* Discussion of the paper by Glennon Gilboy, Jun. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Prof. of Highway Eng., Moscow Superior Technical School, and Moscow Inst. of Transportation Eng., Moscow, Union of the Socialistic Soviet Republics.

‡ Received by the Secretary, July 20, 1928.

§ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 568, Conclusion 7.

‖ *Loc. cit.*, p. 563.

¶ *Loc. cit.*, p. 564.

the clay voids, completely or partly, represents a great percentage of the volume of soil.

The water is even retained in the voids, so that the system mentioned is composed of "soil water". The writer believes that it also includes "entrapped" or "pinched" air which, being held between clay shales, has no communication with the atmosphere and can cause modifications of the elastic properties of the clay.

The structure of clay (or mica) shales is not known exactly. If it is only admitted that these shales are not the last step in the disintegration of rock, then it may be also admitted that shale consists of other thin plates stuck together, with thin air cushions between them. The writer is not at all sure that these ideas are right, but unless the contrary is demonstrated, they cannot be considered as fantastic. If there really is entrapped air between the shale disks and in the interior of the shales, it furnishes an explanation of elastic properties of the clay (and mica), which sands do not possess. Last, but not least, in building foundations (and in a great many highway foundations), soil is not material at all. It is a natural body which the engineer can modify only slightly.

Engineers who study technical properties of soils are also obliged to study their physical properties. Until engineering soil science (physics and mechanics of soils) reaches the present status of metallography in the field of research, the questions "how much" and "why" must be answered simultaneously.*

The writer completely agrees with the statement† that any system of soil analysis which fails to determine the percentage of flat grains can yield no uniformly valid results. In truth, the soil scientist's fastidious methods of mechanical analysis seem to be useless to a practical civil engineer.

The method of representing flat shales of clay to a macroscopic scale is very interesting. Its danger consists, however, in the difference of the behavior of water and air in great and little pores, the physical phenomena in microscopic pores being quite different from those in macroscopic ones. The writer believes, therefore, that the method discussed may be applied to dry soils only, and that the influence of flat particles in wet soils practically has not yet been investigated. It is probable, however, that the solution obtained by the author may be generalized but, strictly speaking, the experiments seem to have failed in attaining their object.

Clay particles being completely different from sand grains as to their shape and properties, the writer suggests that the word, "grains", referring to clay, conveys an inexact idea.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 563.

† *Loc. cit.*, p. 568.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

SILTING OF THE LAKE AT AUSTIN, TEXAS

Discussion*

BY MESSRS. R. F. WALTER, WILLIAM JENNER POWELL, AND R. I. MEEKER.

R. F. WALTER,† M. AM. SOC. C. E. (by letter).‡—The author has furnished the Engineering Profession some interesting and valuable data on the silting of a comparatively small reservoir. The original storage capacity of Lake Austin (49 300 acre-ft.), never represented a very appreciable proportion of the total annual flow of the Colorado River at Austin, Tex. In fact, it was only 1.4% of the average annual flow of 3 500 000 acre-ft. determined by the U. S. Geological Survey for the period from 1898 to 1922.§ With so small a ratio of original storage capacity to total annual run-off on a silt-bearing stream like the Colorado River of Texas it is surprising that the original capacity was not even more rapidly depleted by silt accumulation. However, such a reservoir could hardly be expected to operate efficiently as a desilting agency and undoubtedly large quantities of silt have been carried over the dam during flood periods. It is hoped that Dean Taylor can supply some information regarding this phase of the problem; also, regarding the maximum rates of flood flow and the velocities at which the floods passed through the lake.

The problem of storage depletion by silt accumulation is one that has often been considered by the U. S. Bureau of Reclamation, especially in connection with the investigation of irrigation projects located in the southwestern parts of the United States where the climatic, topographic, and geologic conditions are conducive to rapid rates of soil erosion and correspondingly high percentages of silt content in river flow. Percentage measurements of silt content, by sampling river flow, have been made at several

* Discussion of the paper by T. U. Taylor, M. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Chf. Engr., U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary, August 14, 1928.

§ "Surface Water Supply of the United States, 1922, Pt. VIII, Western Gulf of Mexico Basins," U. S. Geological Survey, *Water Supply Paper 548*, 1925.

stations on the Colorado River and its tributaries above Yuma, Ariz., on the Rio Grande and its tributaries, and on other silt-bearing streams of the Southwest. Actual quantitative measurements of silt accumulated in reservoirs have been made at the Roosevelt, Elephant Butte, and McMillan Reservoirs on the Salt River, Rio Grande, and Carlsbad Projects, respectively. This discussion will present the results of the measurements at the Elephant Butte and McMillan Reservoirs. The operation and maintenance of the Salt River Project was assumed by the Salt River Valley Water Users' Association in November, 1917, and subsequent silt surveys of the Roosevelt Reservoir have been made by that organization.

McMillan Reservoir.—Lake McMillan is located in the Pecos River Valley of Southeastern New Mexico, about fifteen miles from the Town of Carlsbad. The Pecos River rises in the north-central part of New Mexico, on the east side of the Truchas Range of the Sangre De Cristo Mountains. For some distance it is a typical mountain stream, flowing with a swift current through rocky canyons and carrying only such materials as may be eroded and brought in by mountain torrents. These materials settle out rapidly and are seldom transported appreciable distances after they reach the main channel. However, farther down stream, where the course of the river is through alluvial soil, more material is eroded. Below Roswell, tributary streams, draining large areas of arid, mountainous country lying west of the Pecos, bring in heavy loads of silt, so that by the time McMillan Reservoir is reached, the flow is heavily charged. The mean annual flow of the river at Lake McMillan is approximately 316 000 acre-ft., the yearly values varying from a minimum of 130 000 ft. to a maximum of 900 000 acre-ft.

The construction of the original McMillan Dam—a rock-fill structure, 1 686 ft. long and 52 ft. high, with an auxiliary earth embankment, 5 200 ft. long and 19 ft. high—was completed by the Pecos Irrigation Company in the winter of 1893-94. In 1906, following the destruction of the system by floods, reconstruction of the project was undertaken by the U. S. Reclamation Service and since that time it has been successfully operated by the Federal Government. During the period from 1894 to 1903 the reservoir was operated with a spillway crest elevation of 3 258.9 ft., corresponding to a stage of 17.3 ft. on the reservoir gauge. In 1908 and 1909 the outlet works were reconstructed and the embankments repaired so that the lake level could be raised to a stage of 23.5 ft. Further enlargements were made in 1911 and 1912, and again in 1915. The present spillway crest is at Elevation 3 268.6, corresponding to a reservoir stage of 27.0 ft. Reliable topographic data of the original reservoir site were never obtained. However, such information as can be found indicates that the total storage capacity below spillway level, at the time of construction, was about 32 500 acre-ft., or about 10% of the average annual run-off at the reservoir site.

Measurements of silt accumulated in the reservoir were made by the U. S. Geological Survey in 1904, by the U. S. Reclamation Service in 1910 and 1915, and by the U. S. Bureau of Reclamation (formerly the Reclamation Service) in 1925. In the 1904 survey, which was made when the reservoir

was practically empty, depths of silting were determined by auger borings, and the borings were located by plane-table.* Although some uncertainties in determining the elevation of the original ground surface must have been encountered at times, the descriptions of the survey indicate that the work was carefully done and that the final results probably possess a reasonable degree of accuracy. Later measurements of silt accumulations were made by taking cross-sections of the reservoir at intervals throughout the lengths of the silted areas, using customary surveying methods.

Table 8 contains a summary of the data on silt deposited in Lake McMillan as determined by the various surveys, total accumulations in acre-feet below certain stages being given for the periods covered by the investigations. Table 9 gives the average rates of silt accumulation, in acre-feet per year, for the different periods.

TABLE 8.—SUMMARY OF DATA ON SILT ACCUMULATIONS IN LAKE MCMILLAN, CARLSBAD PROJECT, NEW MEXICO.

Time Period.		Duration, in years.	Total Silt Accumulations, in Acre-Feet, Below Reservoir Stages of:			
From.	To.		28.0.	27.0.	25.0.	17.3.
January 1, 1894......	June 1, 1904......	10.42	......	......	18 000	16 000
June 1, 1904......	November 1, 1910......	6.42	......	30 000*	10 000	......
November 1, 1910......	June 1, 1915......	4.58	20 000	18 500	13 400	......
June 1, 1915......	June 1. 1925......	10.00	......	......	3 500	2 500
January 1, 1894......	June 1, 1915......	21.42	......	......	41 400	......
January 1, 1894......	June 1, 1925......	31.42	......	......	44 900	......

* January 1, 1894, to November 1, 1910.

TABLE 9.—RATES OF SILT ACCUMULATION IN LAKE MCMILLAN, CARLSBAD PROJECT, NEW MEXICO.

Time Period.		Duration, in years.	Average Rate of Silt Accumulation, in Acre-Feet, per Year Below Reservoir Stages of:			
From.	To.		28.0.	27.0.	25.0.	17.3.
January 1, 1894......	June 1, 1904......	10.42	......	......	1 730	1 530
June 1, 1904......	November 1. 1910......	6.42	......	1 780*	1 560	......
November 1, 1910......	June 1, 1915......	4.58	4 370	4 040	2 920	......
June 1, 1915......	June 1. 1925......	10.00	......	......	350	250
January 1, 1894......	June 1, 1915......	.....	......	......	1 930	......
January 1, 1894......	June 1, 1925......	31.42	......	......	1 430	......

* January 1, 1894, to November 1, 1910.

The data in Table 8 show that 41 400 acre-ft. of silt were deposited in the reservoir, below the 25-ft. stage, during the period from January 1, 1894, to June 1, 1915, a total approximately one-eighth greater than the entire original capacity of the reservoir below the 17.3-ft. stage. However, the

* Third Annual Report, U. S. Reclamation Service, 1903–04, p. 371.

most interesting feature shown by Tables 8 and 9 is the great reduction in silting which occurred in the reservoir proper during the period from June 1, 1915, to June 1, 1925. During this 10-year period only 3 500 acre-ft. of silt were deposited in the reservoir below the 25.0-ft. stage, giving an average rate of silt deposition of 350 acre-ft. per year as compared with an average rate of 1 930 acre-ft. per year for the period from January 1, 1894, to June 1, 1915. The reason for this marked reduction in rate of silting is that since about 1918 dense and extensive growths of tamarisk (also called salt cedar), have developed at the upper end of the lake. These growths have caused a spreading of the river flow above the reservoir, resulting in velocity reductions and silt deposition, and thus keeping the larger proportions of silt from entering the lake.

The tamarisk is an evergreen shrub, or tree, originally a native of the Mediterranean regions, Western Asia, and India. Its somewhat showy pink and white flowers present an attractive appearance during the blooming period and, consequently, it was brought into this country for use as an ornamental shade tree. Tamarisk trees were propagated in the Pecos River Valley many years before they appeared at Lake McMillan. A clump of rather large trees was growing along the river east of Roswell, about sixty miles above Lake McMillan, some time prior to 1912, when the first tree was noticed at the upper end of the reservoir. The origin of the Lake McMillan growth is not known, but possibly the seed was brought down the river during flood periods, or was brought in by birds. L. E. Foster, Superintendent of the Carlsbad Project, describes the conditions at the upper end of the reservoir, under date of July 20, 1928, as follows:

"At the present time, the entire upper end of the reservoir is covered with a dense growth of tamarisk except for a few narrow channels. This comprises an area of about ten square miles within the flow line of the reservoir. Besides this area the tamarisk growth extends for about two miles up stream, above the reservoir flow line. This area averages about a mile wide in flood time. This flood plane is covered with the same dense growth of tamarisk as characterizes the reservoir area. The nature of the growth changes somewhat from year to year. Where water stands around the plants for more than a week, they usually die. Reforestation begins immediately after the water drains away. At the upper end of the old lake, the growth is quite uniform in age and size but as open water in the lake is approached, the growth varies considerably in size. In many places the growth is so dense as to be almost impenetrable. It ranges in height from about twenty feet down to one foot or less. The diameter of the growth ranges from the size of a pencil to six or eight inches. Just how much effect this comparatively fine, dense growth, together with the down brush, has on the velocities is not certain; but it is certain, however, that the heavier silt deposits are at the upper end of the area where such growth is most dense."

Figs. 17 and 18 are typical views of the tamarisk growth taken near the upper end of the lake on July 16, 1928. The larger trees, such as those along the river banks, are from 15 to 20 ft high. Observations at fence posts, made since 1914, show that as much as 3 ft. of silt has been deposited along the fences in some places, the greater part of which is said to have been deposited during floods since 1915. Perhaps greater depths have been

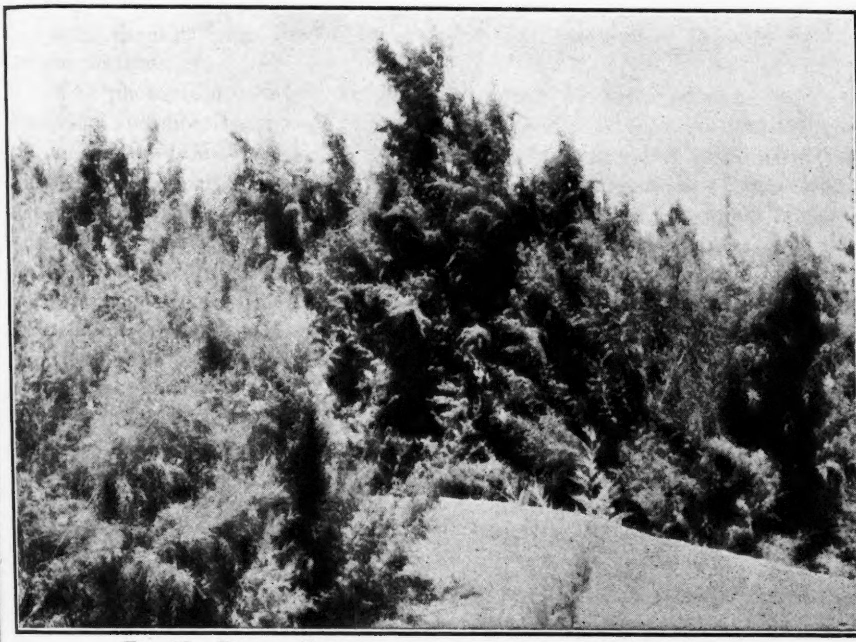

FIG. 17.—TYPICAL TAMARISK GROWTH AT LAKE MCMILLAN, TEXAS.

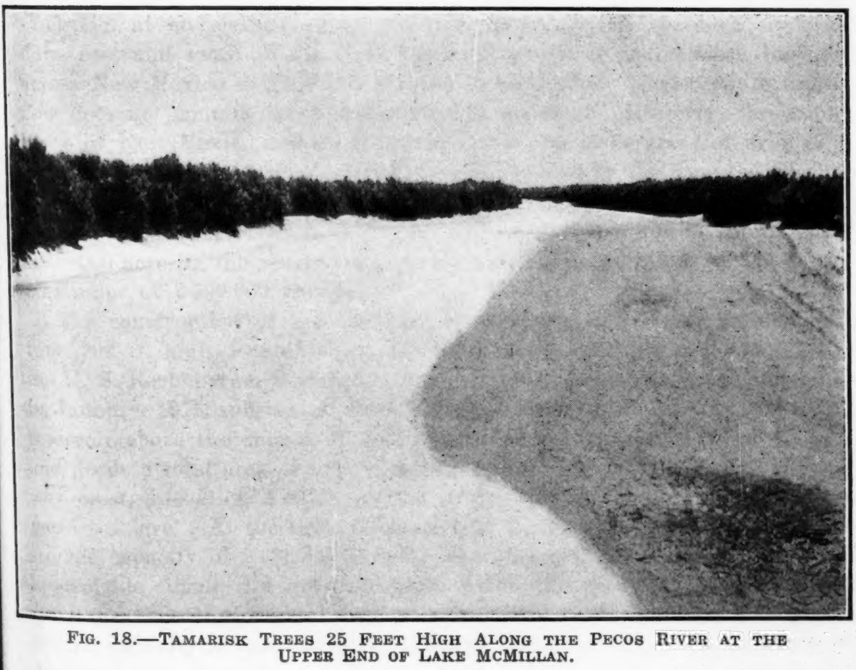

FIG. 18.—TAMARISK TREES 25 FEET HIGH ALONG THE PECOS RIVER AT THE UPPER END OF LAKE MCMILLAN.

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deposited in other places. On account of the impenetrable nature of the growth, the 1925 survey was not extended any appreciable distance into the tamarisk area.

The photographs plainly show that the tamarisk areas at Lake McMillan provide an effective screen for reservoir inflow. It now appears that silt problems at McMillan Reservoir, which were at one time of a serious nature, have been materially lessened by the accidental propagation of this foreign evergreen shrub. Possibly the shrub can be planted at the upper limits of other reservoirs, located in regions of similar climatic conditions, and there utilized to secure similar results. During the summer of 1926 the Middle Rio Grande Conservancy District of New Mexico planted some of the Lake McMillan tamarisk in the Rio Puerco Valley above Socorro, in an attempt to solve the silt problem of the Lower Middle Rio Grande Valley. Some of the plants were alive in the fall of 1927 and bore seed pods, but the pods contained no seed. Possibly the climatic and soil conditions of the Rio Puerco Valley are not favorable to seed production. Tamarisk trees, 20 to 30 ft. high, are growing in Albuquerque, N. Mex., but have never been known to produce seed. However, reproduction by seed takes place farther down the Rio Grande, in the vicinity of El Paso, Tex. Seedlings do not produce seed until about the third year.

Elephant Butte Reservoir.—The Elephant Butte Reservoir, which constitutes the storage unit for the Rio Grande Project of New Mexico and Texas, is located in the Rio Grande Valley of Southern New Mexico. The river rises on the eastern slope of the San Juan Mountains of Southern Colorado, at an elevation approximately 12 000 ft. above mean sea level; flows east and south to the New Mexico State line; then almost due south across New Mexico to El Paso. At the Colorado-New Mexico State line the flow does not contain large proportions of sediment. However, through the State of New Mexico, tributary streams, draining large areas of arid, easily eroded soil, bring in heavy loads of material, so that by the time the Elephant Butte Reservoir is reached the flow is heavily charged with silt. The mean annual flow of the river at the upper end of the reservoir is approximately 1 200 000 acre-ft., the yearly records varying from a minimum of 240 000 to a maximum of 2 500 000 acre-ft.

The construction of the Elephant Butte Dam, a concrete gravity structure, 306 ft. high, located about 130 miles above El Paso, was completed by the U. S. Reclamation Service in 1915. Although some water was impounded in January, 1915, substantial storage did not begin until January, 1916. The reservoir above the dam is 45 miles long, has an average width of 1¾ miles, and floods a total area of approximately 40 000 acres when filled to the spillway crest, Elevation 4 407.0, on the project datum which is 43.3 ft. above mean sea level. At the time of completing the dam the reservoir had a total storage capacity of 2 638 860 acre-ft. below the spillway crest, assuming that appreciable silting did not take place within the flow line of the reservoir from the time the original surveys were made until the dam was finished. Such a capacity is approximately 220% of the mean annual run-off from the drainage area above the reservoir.

A topographic survey of the reservoir site, sufficiently accurate to determine 10-ft. contours, was made below Elevation 4 370 in 1903, and was extended to include the present flow line in 1908. Silt surveys of the upper portion of the reservoir, where practically all the silt is deposited, were made in 1916, 1920, and 1925, measurements being made by running profiles along established cross-sections spaced from ½ to ¾ mile apart. Difficulties were encountered at times in determining the true surface of the silt deposits, due to the comparatively great depths of water, the character of the deposits, and the presence of trees, brush, or débris on the bed of the reservoir. However, the measurements were made with a great deal of care and the results are probably sufficiently accurate for the purposes intended. An exhaustive study of the chemical and physical properties of the silt was made in 1916.*

The results of the 1920 survey, as compared with the results of the 1916 survey, showed that 90 858 acre-ft. of silt were deposited in the reservoir during the intervening period of 48.7 months, corresponding to an average rate of deposition of 1 865 acre-ft. per month, or 22 400 acre-ft. per year. Comparison of the results of the 1920 survey with the original reservoir topography, taken in 1903 and 1908, indicated a total silt accumulation of 140 000 acre-ft. up to the time of the 1920 survey, practically all of which must have taken place after the reservoir began storing water. Assuming that storage began in January, 1915, the corresponding rate of silting would be 2 060 acre-ft. per month, or 24 700 acre-ft. per year. However, the Project Manager considered the values based on the original topography to be less accurate than those based on the 1916 survey. The total silt accumulation indicated by the 1916 and 1920 surveys amounted to 1.66% of the total inflow to the reservoir during the intervening period.

TABLE 10.—Silt Deposition at Elephant Butte Reservoir.

Elevation, in feet.*	Original reservoir capacity, in acre-feet.	1925 Conditions.		Period, 1916 to 1925.†		
		Total silt deposited, in acre-feet.	Remaining reservoir capacity, in acre-feet.	Total silt deposited, in acre-feet.	Rates of Silting in: Acre-feet per year.	Rates of Silting in: Acre-feet per month.
4 407	2 638 860	231 735	2 407 125	177 740	20 508	1 709
4 400	2 365 058	225 287	2 139 771	172 711	19 928	1 661
4 390	2 009 384	213 221	1 796 163	162 930	18 800	1 567
4 380	1 693 971	201 297	1 492 674	152 870	17 639	1 470
4 370	1 406 600	179 015	1 227 585	128 985	14 883	1 240
4 360	1 163 188	148 312	1 014 876	95 746	11 048	921
4 350	953 041	127 539	825 502	78 144	9 017	751
4 340	776 741	113 519	663 222	65 101	7 512	626
4 330	620 323	100 156	520 167	52 684	6 079	507
4 325	556 100	91 993	464 107	49 945	5 763	480

* Above project datum, add 43.3 ft. for sea-level elevations.

† Total time, 104 months.

Table 10 gives data on reservoir capacities and silt accumulations below elevations varying from 4 325 to 4 407 ft., as determined by the 1925 survey. Fig. 19 shows the effect of silting on the capacity of the reservoir.

* "Notes on Silting of Elephant Butte Reservoir," *Reclamation Record*, September, 1916, p. 423.

The results of the 1925 survey indicate a total silt accumulation of 231 735 acre-ft. from the time the original reservoir topography was taken to the time of the 1925 survey, and a total accumulation of 177 740 acre-ft. during the 104 months from the time of the 1916 survey. The latter figure corresponds to an average monthly deposit of 1 709 acre-ft. and an average yearly deposit of 20 508 acre-ft. It also corresponds to 1.64% of the total reservoir inflow during the period considered, a figure almost the same as that obtained by

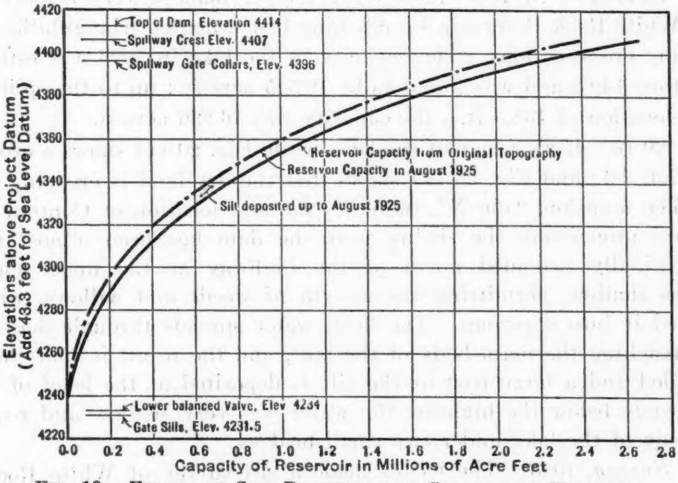

FIG. 19.—EFFECT OF SILT DEPOSITION ON CAPACITY OF ELEPHANT BUTTE RESERVOIR.

the 1920 survey. Thus far, the investigations show that 72%, or nearly three-fourths, of the silt that enters the reservoir is deposited within an area containing not quite one-fourth of the total reservoir capacity. The total accumulation of 231 735 acre-ft. shown by the 1925 survey is only about 8.78% of the original storage capacity below the spillway crest, indicating an average yearly rate of silting of slightly less than 1% of the original storage capacity. Assuming such a silting rate to keep up indefinitely (which would be a severe assumption), the total life of the reservoir would be 114 years. Since 13 years have already passed, the remaining life would be 101 years.

WILLIAM JENNER POWELL,* M. AM. SOC. C. E. (by letter).†—The writer has known in a general way that Lake Austin was silting rapidly, but he was not prepared for the staggering information‡ that 95% of the water capacity had been taken up by silt in thirteen years of service. The water-shed of the Colorado River for about 38 000 sq. miles spreads over what is generally known as a grassy or wooded area including, in its topographical features, flat prairie country and rugged mountainous sections.

Recently the writer made a survey of the silting of the White Rock Reservoir, which is about four miles east of the City Hall of Dallas, Tex. The

* Cons. Engr., Dallas, Tex.

† Received by the Secretary, August 16, 1928.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 575.

drainage area is a wide limestone valley 114 sq. miles in extent. The reservoir was begun in 1910 and was ready for service in 1912. Practically the lowest point in the lake was 424 ft. above mean low tide. Careful surveys were not made of the original volume of water and its capacity was only estimated. The dam is of earth and a concrete spillway with a long wide apron was constructed at the east end at elevation 138.5, City of Dallas datum, or Elevation 457.45 above mean low tide.

Silt Surveys.—In 1923, the City Engineer made a very elaborate survey of the White Rock Reservoir, establishing bench-marks, triangulation stations, and many cross-sections. The capacity of the lake in 1923 was estimated up to Contour 140.5 and was found to be 19 525 acre-ft.; up to the spillway crest (local elevation of 138.5 ft.), the capacity was 16 896 acre-ft.

Fig. 20 (*a*) shows a map of the lake, while Fig. 20 (*b*) shows a cross-section at Section 2-4; and Fig. 20 (*c*) shows that the dry land is creeping down the lake. The notation, "136 N", indicates the new location of Contour 136. As a general proposition the silting near the dam has been almost nil. The lake originally extended north of the highway at the upper end. This part was shallow, permitting the growth of weeds and willows, which soon converted it into a swamp. The flood water spreads through these marshes before reaching the main body of the lake, and the result is that the current is retarded and a large part of the silt is deposited at the head of the lake. Immediately below the highway the water was very shallow and reached the main body of the lake under two small bridges.

Silt Survey, 1928.—The writer made a silt survey of White Rock Reservoir in July, 1928, with soundings at Sections 4-2, 1-2, 3-4, 5-6, 7-8, 9-10, 11-12, and 12-13 (Fig. 20 (*a*)). These soundings were compared with those taken in 1924 by Messrs. McCombs and Ernest of the City Engineer's Office of Dallas. No evidence of any silt was found at Section 5-6 or between Section 5-6 and the dam. Decided evidence of silt was found from Section 3-4 to the head of the lake. The maximum depth of water in Section 3-4 was found to be 5.6 ft. below Contour 140.5, or 3.6 below Contour 138.5.

At Section 3-4, the following results were found:

		Mean depth, in feet.
Area, 1923	= 14 400 sq. ft.	6.00
" , 1928	= 6 600 " "	2.75
Silt area	= 7 800 sq. ft.	3.25

At Section 1-2:

Area, 1923	= 6 630 sq. ft.	5.1
" , 1928	= 4 810 " "	3.7
Silt area	= 1 820 sq. ft.	1.4

Attempts were made to take readings along Section 0-2, when the lake level was 0.1 ft. below the crest of the spillway, but nearly all the section was dry land or boggy marsh. It became necessary to take soundings along Section 2-4, and at that section the following results were found:

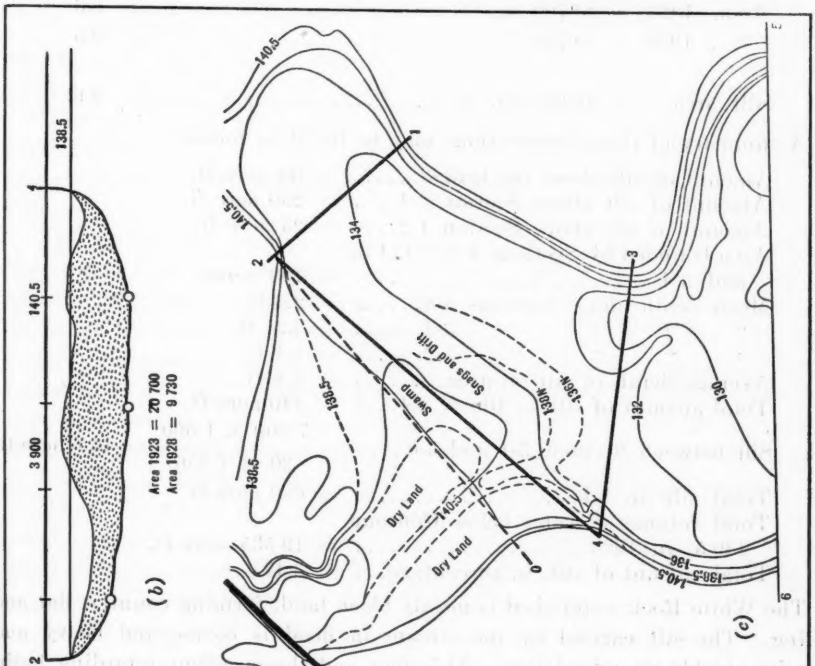

FIG. 20.—WHITE ROCK RESERVOIR, DALLAS, TEXAS.

		Mean depth, in feet.
Area, 1923	= 20 700 sq. ft.	5.3
" , 1928	= 9 730 " "	2.5
Silt area	= 10 970 sq. ft.	2.6

A summary of these observations may be listed as follows:

Amount of silt above the bridge	= 30 acre-ft.
Amount of silt above Section 2-4	= 250 acre-ft.
Amount of silt above Section 1-2	= 25 acre-ft.
Area bounded by Sections 4-2, 2-1, 1-3, and 3-4	= 100 acres.
Mean depth along Sections 4-2	= 2.6 ft.
" " " " 3-4	= 3.25 ft.
" " " " 1-2	= 1.4 ft.
Average depth of silt on area	= 2.4 ft.
Total amount of silt = 100 × 2.4	= 240 acre-ft.
Silt between Sections 3-4 and 5-6	$= \dfrac{7\,800 \times 1\,500}{20 \times 4\,356} = 135$ acre-ft.
Total silt in lake	= 680 acre-ft.
Total volume of water below Contour 140.5, in 1923	= 19 535 acre-ft.
Total amount of silt, in percentage	= 3.47

The White Rock water-shed is mostly black land, farming country, flat and rolling. The silt carried by the stream in flood is coarse and heavy and deposits quickly on retardation. This fact and the excellent retarding basin formed by the growth at the head of the lake have practically prevented silting in the main body of the lake, and by making land in the shallows at the upper end rather than distributing the silt over the lake bottom, the silting has been more gain than loss, since it has made usable for park purposes a large area that was formerly marshy and valueless. This is of interest in view of the fact that the lake is shortly to be abandoned as a source of water supply and the entire property is expected to be devoted to park and recreational purposes.

R. I. Meeker,* M. Am. Soc. C. E. (by letter).†—This paper is of more than passing interest to hydraulic engineers. Demands of the future for irrigation, power, and flood control will compel greater use of channel reservoir sites. Reservoir silting at Austin, Tex., Zuni, Elephant Butte, and McMillan,‡ New Mexico, Roosevelt, Arizona, and projects in adjacent States, should awaken State and Federal departments to the need of a survey and scientific study of erosion and means of control. If emphasis is needed the following storage capacity losses from silt deposition should prove convincing.

In December, 1925, with but eleven years of operating history, the capacity table of the Elephant Butte Reservoir on the Rio Grande was corrected for a loss of 220 000 acre-ft., due to silt deposition. At this rate the capacity will be reduced to 1 418 000 acre-ft., by 1975, or 54% of the original capacity of

* Cons. Engr., Denver, Colo.

† Received by the Secretary, August 24, 1928.

‡ Third Report, U. S. Reclamation Service, 1903-04, pp. 371-373.

2 638 000 acre-ft. The drainage area of the Elephant Butte Reservoir is more than 30 000 sq. miles.

At the Roosevelt Reservoir,* which had an original capacity of 1 367 000 acre-ft., a silt deposition of 101 000 acre-ft. is reported during a 20-year period ending in 1925. The drainage area of the reservoir is 5 760 sq. miles.

Examples of heavy silting in small channel reservoirs occur at the Castlewood Reservoir, on Cherry Creek, near Denver, Colo., and the Buckhorn Reservoir, on a small tributary of the Big Thompson River, near Loveland, Colo.

The Castlewood Dam was completed in 1890. The reservoir originally impounded 5 267 acre-ft. After thirty-eight years of operation the reservoir capacity is greatly reduced; precise data, as yet, are not available as to the decreased capacity. The Castlewood Reservoir is located in the foothills on a plains tributary of the South Platte River, where the average annual precipitation is 18 in. The drainage area is 165 sq. miles. The Buckhorn Reservoir was completed in 1907. The original capacity was 1 191 acre-ft. In 1925, eighteen years later, a re-survey showed that silt deposition had reduced the capacity to 626 acre-ft., or practically one-half the original capacity. The Buckhorn Reservoir is located at the edge of the mountains where the annual precipitation is 16 in. The drainage area above the reservoir is 130 sq. miles. Both the Castlewood and Buckhorn Reservoirs are fed by streams subject to so-called "cloudburst" storms.

Rising river beds of the Rio Grande near San Marcial, N. Mex., and the Arkansas in Western Kansas, are also indicative of the heavy sand and silt burden resulting from erosion induced by settlement. The former has risen 12 ft., and the latter 5 ft., in a period of 45 years.

Considerable information is now available on quantities of silt carried by streams of the Southwest and on silt deposition in reservoirs thereon. The origin of the silt question is erosion, and the writer feels that the problem should be attacked from that angle. Information is needed on its causes and control. Over-grazing by sheep and cattle has heavily depleted the grass carpet of the plains, foot-hills, and plateau areas of the Colorado, Rio Grande, Arkansas, and Platte River Basins and has contributed greatly to erosion, which is increasing in the arid Southwest. Enormous areas of public lands are affected, and public agencies cannot ignore the problem of erosion when its far-reaching effect on future water control and utilization is realized. Erosion impinges heavily on the present and future water problems of the Western United States. Experience in arid South Africa confirms the experience in the Southwestern United States, and the following statement† appears in agreement with American history:

"In course of time burning and the concentrated grazing and tramping of sheep, goats, and cattle caused progressive desiccation. To-day, the broad valleys are drained by deep and wide river channels, and the rich grass flats

* "Silt in the Colorado River and Its Relation to Irrigation", by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin 67*, U. S. Dept. of Agriculture, 1928, p. 22.

† "The Principles of Irrigation Engineering", by F. E. Kanthack, pp. 103–105.

are dried out and eroded. Rivers only a few feet wide a generation ago are now two or three hundred feet in width. The dense bush in the valleys has disappeared, and the river channels, now ten or twenty times the size they used to be, present an unobstructed passage to flood water entering them."

The recent *Bulletin** on "Silt in the Colorado River", by the U. S. Department of Agriculture, could well be a forerunner of another *Bulletin* on erosion. Through the Forest Service and associated divisions, the Department of Agriculture is well qualified to study this important problem in a comprehensive and scientific way.

The present investigational studies of erosion and silt control by the Middle Rio Grande Conservancy District in New Mexico is intelligent pioneering on a subject destined to play an important part in present and future water conservation. The States of Arizona, Colorado, New Mexico, Texas, Utah, and Wyoming cannot afford to ignore the problem of erosion and means for its control. The Colorado River Basin and the Rio Grande Basin offer excellent opportunities for a systematic survey and study of that problem.

TABLE 13.—SILT DEPOSITION, CHANNEL RESERVOIRS, SOUTHWESTERN UNITED STATES.

Reservoir.	Stream.	State.	Drainage area, in square miles.	Original capacity, in acre-feet.	Silt Deposition.		
					Lost capacity, in acre-feet.	Period, in years.	Percentage of original capacity.
Elephant Butte...	Rio Grande	New Mexico.	30 000	2 368 000	220 000	11-1915-1925	9
Roosevelt	Salt..............	Arizona......	5 760	1 367 000	101 000	20-1906-1925	7
Austin............	Colorado	Texas........	38 000	*49 300	23 559	8-1893-1900	48
				32 029	30 552	13-1913-1926	95
McMillan..........	Pecos............	New Mexico.	......	28 732	12 232	10-1895-1904	42
Zuni..............	Zuni	New Mexico.	500	10 230	9 689	22-1905-1927	95
Castlewood.......	Cherry Creek....	Colorado	165	5 267	†	38-1890-1927	†
Buckhorn.........	Buckhorn Creek	Colorado	130	1 191	565	18-1907-1925	47

* Old reservoir.

† Precise data are not available at this time.

Grazing control on the Jornado Grazing Reserve north of Las Cruces, N. Mex., may develop some valuable information concerning erosion. An interesting bibliography on silt problems, prepared by the Special Committee on Irrigation Hydraulics, has been published by the Society.† Table 13 summarizes the losses in various reservoirs from silt deposition in the Southwest.

* *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, 1928.

† *Proceedings*, Am. Soc. C. E., March, 1925, Society Affairs, pp. 151–153.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

CONTINUOUS BEAMS OVER THREE SPANS

Discussion*

By H. M. Hadley, Assoc. M. Am. Soc. C. E.

H. M. Hadley,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author's discussion of and comments on the practical present-day methods of designing continuous beams very happily describes and appraises them. They are arbitrary; even capricious. The only thing to be said in their favor is that they are simple, and that things are accomplished by them. The designer looks at his spans and his loads; considers for a moment; decides that $w\,l$ over something is about right; and forthwith proceeds with his computations. An experienced man will form judgments that may quite closely approximate the theoretical stress distribution; the judgments of the inexperienced man will not be so close. In either event some definite result will be obtained and some decision reached that will find expression in designated sizes of beams, specified quantity of reinforcement, or similar directions. Such designs, when built and submitted to the actual test of gravity, will in most cases appear to be satisfactory. Nothing alarming happens; a few cracks may develop; nothing more. The construction stands up and carries applied loads without pronounced signs of distress, and, in the busy rush of affairs, with these results the designers are, perforce, content.

Probably no one is wholly pleased with such methods of design, but in the absence of reasonably simple and economical means of obtaining a correct solution of the problem, the arbitrary and expeditious ways of designing continuous beams will doubtless continue to find favor.

Mr. Oesterblom's selection of the continuous beam of three spans as the one of outstanding importance is well taken. It is of more frequent occurrence than any other, and in multi-storied, typical construction that is repeated over and over again in the same building, this specific problem, if any,

* Discussion on the paper by I. Oesterblom, M. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Dist. Engr., Portland Cement Assoc., Seattle, Wash.

‡ Received by the Secretary, May 12, 1928.

is deserving of a correct solution, because the economies that may be effected thereby are multiplied manyfold. Similarly, such buildings offer opportunities for repeated mistakes.

In giving separate consideration to dead and live loads and in treating the spans individually, determining the separate moment effects, and combining them for maximum conditions, Mr. Oesterblom's treatment is most excelcellent and logical.* The influence of loads on adjacent spans is made clear and plain by this method, and one can see the influence of the several parts in the combined result.

The statement that moment factors are not properly a subject for regulation by anything akin to a code has an engaging interest. King Canute and the rising tide come to mind. Will the day come when $\frac{w\,l^2}{12}$ will no longer be established and ordained in American building laws?

As to the moment factors in Table 3,† they apply exactly to uniform loads and symmetrical span arrangements; but as loads depart from uniformity and spans from symmetry, so do these moment factors depart from true and proper values. Why "moments due to equi-distant concentrated loads are so nearly equal to uniform loads that the same moment factors would apply",‡ is not wholly apparent. The adaptation of these moment factors to the case of different moments of inertia in the different spans is not a matter of simple facility. Neither is it clear why an attempt should be made to develop moment factors except for the supports. With the moments at the supports known, the stress distribution elsewhere becomes a simple problem, and it is only for uniform loads that these mid-span factors apply at all.

These difficulties seem to be inherent in the approach to the problem by the Theorem of Three Moments. Mr. Oesterblom himself recognizes the trouble in his statement§ that "clearly it is impracticable to make a table of moment factors for all possible and reasonable span combinations." He states that 2 025 combinations with 15 moment factors each would be required to cover spans varying from 8 ft. to 30 ft. by 6-in. increments. It is many times as impracticable if, to the moment factors for all possible and reasonable span combinations, are added the moment factors for all possible and reasonable load combinations.

From another angle of approach, however, all possible and reasonable span combinations together with all possible and reasonable load combinations become subject to an extremely easy and simple treatment. A method of "unbalanced moments", developed by the writer, which is a special application of the slope deflection method, makes it possible to derive coefficients for the support moments, which are as true and "universal" as Equation (1).‖

Such coefficients, for the author's case of a continuous beam of three spans, with free ends and unyielding supports, are given in Fig. 6. They are in algebraic form and are expressed in terms of the "stiffness factors" of the

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 718.

† *Loc. cit.*, p. 719.

‡ *Loc. cit.*, p. 711.

§ *Loc. cit.*, p. 717.

‖ *Loc. cit.*, p. 710.

three spans. For a constant value of the modulus of elasticity, these "stiffness factors" are simply the values obtained by dividing the moment of inertia of each span member by the span length, that is, $K = \frac{E\,I}{l}$. Fortunately, it is the relative and not the absolute value of these "stiffness factors" that is of importance and affects the stress distribution in the system. The derivation of these coefficients will not be presented herein, but the method of their use will be briefly stated and they will be applied to two problems.

The successive steps in their application are as follows:

(1) Determine the relative "stiffness factors" of the several spans.

(2) Substitute these values in Fig. 6 and obtain the numerical coefficients that are applicable to the particular problem under consideration.

(3) Consider each span of the continuous beam as a separate, individual, single span with fixed ends. Compute the end moments which would result from the loads upon the span under this assumption.

(4) Next, imagine these separate and individual fixed-end spans with their end moments to be joined and connected, to form the three-span continuous beam with free ends. Following this joining of the separate, individual, fixed-end spans, a complete re-adjustment of stress from that assumed under Step (3) occurs throughout the system. At Support (1), the free end of the continuous beam, there is nothing to resist or balance the moment which would develop there if Span 12 were the separate, fixed-end span that has previously been assumed. Consequently, this moment, unbalanced, must change and fall to zero, but in so doing it changes the stress distribution and the elastic curve of the beam in all spans. For a moment at any support to change its value implies a movement or rotation at that point, which movement and elastic adjustment carries out to the extremities of the system.

At Support (2), Spans 12 and 23 meet and are joined. Each of these spans had end moments under the assumption in Step (3). These end moments are opposite in direction—clockwise and counter-clockwise—and although they may be equal and may exactly balance one another, are generally unequal, either one side or the other being larger. It is the excess moment, the amount by which one side is greater than the other, that is unbalanced, and that, in changing, lessening, and developing a resistance, produces a re-adjustment of stress throughout the entire system. As for the equal, opposite, and balanced moments—the full end moment of one span, and its equal and opposite part of the end moment of the other span—these are balanced and create no change in stress anywhere in the system, regardless of their value or magnitude. Follow a similar procedure at Supports (3) and (4).

(5) The final step is to calculate the balanced moments at each support, and the moment effects produced at each support by the several unbalanced moments. The algebraic sum of these quantities gives the actual moments existing at the supports. In this particular case, that of the three-span continuous beam, the moments at Supports 2 and 3 are determined. When these are known, the stress distribution in each span may be found by simple statics.

The coefficients give the moment effects at all supports produced by the stress adjustment of an unbalanced unit moment at any of the supports. These coefficients are abstract numbers and whether moments are measured in inch-

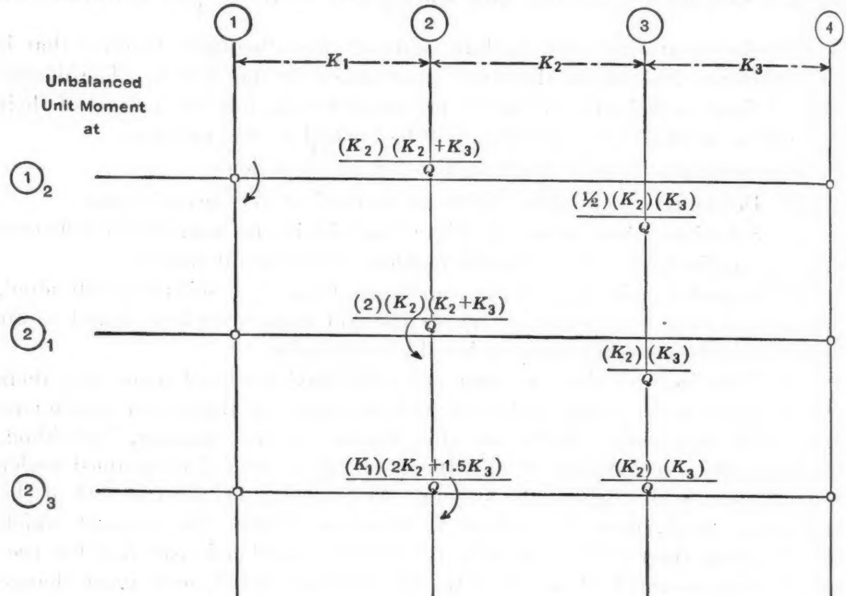

FIG. 6.—VALUES OF MOMENTS AT SUPPORTS DUE TO STRESS ADJUSTMENTS OF UNBALANCED UNIT MOMENTS.

pounds or kip-feet is immaterial. Having found the unbalanced moment at any support, multiply the corresponding coefficients by the value of this unbalanced moment, and the final moment effects at all supports due to this unbalanced moment will be found. Attention should be drawn to the fact

that the coefficients change with a change of the unbalanced moment from one side of a support to the other; also, that the coefficients are written on the tension side of the beam. If the effect is to produce tension on the top of the beam, the coefficient is written above the line; if tension is on the bottom, the coefficient is written below the line. The unbalanced unit moment of Fig. 6 is assumed to produce tension on the top of the beam. Should the case arise in any problem of an unbalanced moment producing tension on the bottom of the beam, the natural consequent inversion of resulting values should be made.

In tabulating the moment effects at the supports and in combining them with the balanced moments, it is necessary to add positive and negative values. Therefore, as in the two illustrative problems, the convention is adopted of writing the moments producing tension on the top of the beam at the left of the vertical lines, and those producing tension on the bottom, at the right. This is an arbitrary convention and the position of a balanced moment as tabulated for addition may be opposite the position in which it appears in the line of fixed-beam moments.

Example 1.—Consider the author's Example 1,* which has a 9-ft. corridor span and two outside spans of 22 ft. 6 in. each; 67 lb. dead load; 60 lb. live load on outer spans and 80 lb. live load on corridor; and equal moments of inertia in all spans.

Then,

$$K_1 = \frac{I}{22.5} = 1.0 \text{ (relatively)}$$

$$K_2 = \frac{I}{9} = 2.5$$

$$K_3 = \frac{I}{22.5} = 1.0$$

Substituting these values in the algebraic coefficients given in Fig. 6, the results are as shown in Table 8. The numerical solution of Example 1 is given in Table 9.

TABLE 8.—NUMERICAL COEFFICIENTS.

Unbalanced unit moment at:	Support (1).	Support (2.	Support (3).	Support (4).
Support $(1)_2$	0	0.365	0.0521	0
Support $(2)_1$	0	0.730		0
Support $(2)_3$	0	0.270	0.1042 0.1042	0

The maximum "positive" moment in Span 12 will occur when the moment at Support (2) is a mimimum. By inspection of the values found, this will occur with dead load on all spans and live load on Spans 12 and 34. With this loading the moment at Support (2) is 62 410 in-lb.

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.

For the case of a beam of span length, l, subjected to a uniform load, w lb. per ft., with a moment, M, applied at one end, the distance of the inflection point from the end at which the moment is applied is given by Equation (34):

$$X\,(\text{feet}) = \frac{2\,M}{l} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)^{*}$$

TABLE 9.—NUMERICAL SOLUTION OF EXAMPLE 1.

Support No.	(1)	(2)		(3)		(4)
Span length, in feet	22.5		9.0		22.5	
UNIFORMLY DISTRIBUTED DEAD LOAD OF 67 POUNDS PER FOOT ON ALL SPANS						
Fixed beam bending moment	33.92*	33.92	5.43	5.43	33.92	33.92
Unbalanced bending moment	33.92	28.49	....	....	28.49	33.92
Balanced bending moment	0	+ 5.43	....	+ 5.43	....	0
Unbalanced moment at Support $(1)_2$	0	+12.38	....	....	− 1.77	0
Unbalanced moment at Support $(2)_1$	0	+20.80	....	....	− 2.96	0
Unbalanced moment at Support $(3)_4$	0	.....	−2.96	+20.80	....	0
Unbalanced moment at Support $(4)_3$	0	.....	−1.77	+12.38	....	0
Summation of unbalanced bending moments	0 ..	+38.61 − 4.73	−4.73	+38.61 − 4.73	− 4.73	0 ..
Actual moments	0	+33.88	....	+33.88	....	0
UNIFORMLY DISTRIBUTED LIVE LOAD ON SPAN 12 EQUAL TO 60 POUNDS PER FOOT						
Fixed beam bending moment	30.38	30.38	0	0	0	0
Unbalanced bending moment	30.38	30.38	0	0	0	0
Balanced bending moment	0	0	..	0	..	0
Unbalanced moment at Support $(1)_2$	0	+11.09	..	..	− 1.58	0
Unbalanced moment at Support $(2)_1$	0	+22.18	..	..	− 3.16	0
Actual moments	0	+33.27	..	..	− 4.74	0
UNIFORMLY DISTRIBUTED LIVE LOAD ON SPAN 23 EQUAL TO 80 POUNDS PER FOOT						
Fixed beam bending moment	0	0	6.48	6.48	0	0
Unbalanced bending moment	0	..	6.48	6.48	..	0
Balanced bending moment	0	0	..	0	..	0
Unbalanced moment at Support $(2)_3$	0	+ 1.75	..	+ 0.67	..	0
Unbalanced moment at Support $(3)_2$	0	+ 0.67	...	+ 1.75	..	0
Actual moments	0	+ 2.42	..	+ 2.42	.	0
UNIFORMLY DISTRIBUTED LIVE LOAD ON SPAN 34 EQUAL TO 60 POUNDS PER FOOT						
Actual moments	0	− 4.74	..	....	+33.27	0

* All bending moments are given in kip-inches.

* M and l must be given in same linear units.

Between the free end and the inflection point, the stress distribution is that of a simple beam of load, w, and span, $l - x$. Therefore, the maximum positive moment in Span 12 is as follows:

Inflection point:

$$x = \frac{2 \times 62\,410}{127 \times 270} = 3.64 \text{ ft.}$$

New span:

$$22.5 - 3.64 = 18.86$$

$$M = 127 \times 355.7 \times 1.5 = 67\,600 \text{ in-lb.}$$

At Support (2) the maximum moment occurs with dead load on all spans and live load on Spans 12 and 23; combining, this is 69 570 in-lb.

At mid-span, in Span 23, the "negative" moment will be greatest with dead load on all spans and live load on Spans 12 and 34. With such loads the moments at Supports (2) and (3) are 62 410 in-lb. Then the moment at mid-span is,

$$62\,410 - 67 \times 81 \times 1.5 = 54\,270 \text{ in-lb.}$$

Support (3) and Span 34 are determined by symmetry. These values all agree closely with those found by the author.

Example 2.—A certain large apartment house in Seattle, Wash., has fourteen floors with spans of 22 ft. 6 in., 7 ft. 0 in., and 18 ft. 0 in.; dead load of 70 lb. per ft.; live load of 55 lb. per ft.; and relative $I_s = 5$, 1, and 5. The "stiffness factors":

$$K_1 = \frac{5}{22.5} = 0.222 = 1.6 \text{ (relatively)}$$

$$K_2 = \frac{1}{7} = 0.143 = 1.0$$

$$K_3 = \frac{5}{18} = 0.278 = 2.0$$

The numerical coefficients are as given in Table 10. The numerical solution of Example 2 is given in Table 11.

TABLE 10.—NUMERICAL COEFFICIENTS.

Unbalanced moment at:	Support (1).	Support (2).	Support (3).	Support (4).
Support $(1)_2$	0	0.214	0.071	0
Support $(2)_1$	0	0.429	0.143	0
Support $(2)_3$	0	0.571	0.143	0
Support $(3)_2$	0	0.114	0.628	0
Support $(3)_4$	0	0.114	0.372	0
Support $(4)_3$	0	0.057	0.186	0

As in Example 1, the moments may be determined from Table 11 to be as follows:

Moment at Support (2), maximum.............. 40 900 in-lb.
" " " (2), for maximum mid-span moment in Spans 12 and 23.............. 36 100 in-lb.
Moment at Support (3), maximum.............. 19 700 in-lb.
" " " (3), for maximum mid-span moment in Spans 23 and 34.............. 11 600 in-lb.

The maximum mid-span moment in Span 12 is:

Inflection point:

$$x = \frac{2 \times 36\,100}{125 \times 270} = 2.14 \text{ ft.}$$

New span:

$$22.5 - 2.14 = 20.36$$

$$M = (70 + 55) \times 20.36^2 \times 1.5 = 77\,700 \text{ in-lb.}$$

and the maximum mid-span moment in Span 34 is:

Inflection point:

$$x = \frac{2 \times 11\,600}{125 \times 216} = 0.86 \text{ ft.}$$

New span:

$$18.0 - 0.86 = 17.14$$

$$M = 125 \times 17.14^2 \times 1.5 = 55\,100 \text{ in-lb.}$$

In the case of the mid-span moment in Span 23, there are moments at both ends, 36 100 and 11 600 in-lb. The excess of the larger moment at Support (2) over the smaller moment at Support (3) is 24 500 in-lb. Ordinarily, for the determination of the maximum or minimum values of the mid-span moment, this excess moment is treated precisely like the end moments in Spans 12 and 34 for finding an inflection point and new span length, etc., and, after this, a final correction is made for the smaller end moment, the value of which is applied as a constant to the results obtained. Here, however, the excess moment is so great that there is no inflection point, and no maximum or minimum value of the mid-span moment. If it is assumed that the mid-span moment is the average of the end moments, the assumption will closely approximate the truth, and any error involved will be on the safe side.

These examples have dealt with uniform loads. Any other system of loads—concentrated or variable—for which fixed-beam end moments can be calculated, can be treated with equal ease, since the method deals with unbalanced moments and not directly with the beam loads. Variable span lengths and different moments of inertia of the several spans can be handled with practically the same facility as equal spans and constant moments of inertia. The effect of settlement of supports likewise can be included in the calculations with these same coefficients in an extremely simple and ready manner.

For other than three spans, and for the cases of free ends, fixed ends, one end free, and one fixed, and of variable amounts of restraint at the ends, similar tables of coefficients can be prepared and thereby the continuous beam (continuous also in the interminable length of its required calculations)

TABLE 11.—NUMERICAL SOLUTION OF EXAMPLE 2.

Support No.	(1)		(2)		(3)	(4)
Span length, in feet		22.5		7.0		18.0
UNIFORMLY DISTRIBUTED DEAD LOAD OF 70 POUNDS PER FOOT ON ALL SPANS						
Fixed beam bending moment	35.4*	35.4	3.4	3.4	22.7	22.7
Unbalanced bending moment	35.4	32.0	...	...	19.3	22.7
Balanced bending moment	0	+ 3.4	...	+ 3.4	....	0
Unbalanced moment at Support $(1)_2$	0	+ 7.6	...	...	− 2.5	0
Unbalanced moment at Support $(2)_1$	0	+13.7	...	...	− 4.6	0
Unbalanced moment at Support $(3)_4$	0	....	−2.2	+ 7.2	....	0
Unbalanced moment at Support $(4)_3$	0	....	−1.3	+ 4.2	....	0
Summation of unbalanced bending moments	0	+24.7	−3.5	+14.8	− 7.1	0
	..	− 3.5	...	− 7.1	....	..
Actual moments	0	+21.2	...	+ 7.7	....	0
UNIFORMLY DISTRIBUTED LIVE LOAD ON SPAN 12 EQUAL TO 55 POUNDS PER FOOT						
Fixed beam bending moment	27.8	27.8	0	0	0	0
Unbalanced bending moment	27.8	27.8	0	0	0	0
Balanced bending moment	0	....	0	0	....	0
Unbalanced moment at Support $(1)_2$	0	+ 6.0	..	..	−2.0	0
Unbalanced moment at Support $(2)_1$	0	+11.9	..	..	−4.0	0
Actual moments	0	+17.9	..	..	−6.0	0
UNIFORMLY DISTRIBUTED LIVE LOAD ON SPAN 23 EQUAL TO 55 POUNDS PER FOOT						
Fixed beam bending moment	0	0	2.7	2.7	0	0
Unbalanced bending moment	0	..	2.7	2.7	..	0
Balanced bending moment	0	0	...	0	..	0
Unbalanced moment at Support $(2)_3$	0	+1.5	...	+0.4	..	0
Unbalanced moment at Support $(3)_2$	0	+0.3	...	+1.7	..	0
Actual moments	0	+1.8	...	+2.1	..	0
UNIFORMLY DISTRIBUTED LIVE LOAD ON SPAN 34 EQUAL TO 55 POUNDS PER FOOT						
Fixed beam bending moment	0	0	0	0	17.8	17.8
Unbalanced bending moment	0	..	0		17.8	17.8
Balanced bending moment	0	0	...	0	....	0
Unbalanced moment at Support $(3)_4$	0	..	−2.0	+6.6	....	0
Unbalanced moment at Support $(4)_3$	0	..	−1.0	+3.3	....	0
Actual moments	0	..	−3.0	+9.9	....	0

* All bending moments are given in kip-inches.

becomes simple indeed. With a simple and easy method available for calculating results, the designer can vary the moments of inertia of his members, and thereby obtain a control over the distribution of stresses in his continuous beam that heretofore has been attempted only by varying the span lengths.

While he can do this, it is to be recognized, of course, that "The Building Code", or something equally authoritative, will have its final say in the matter, and, again, the distinction between "can" and "may" is to be acknowledged and admitted—by Man, if not by Nature.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

FLOOD CONTROL ON THE RIVER PO IN ITALY

Discussion*

By Messrs. M. Giandotti, R. D. Goodrich, and C. E. Grunsky.

M. Giandotti,† Esq. (by letter).‡—This paper describes very well the actual conditions—geo-physical, morphological, and hydraulic—that occur on the River Po.

The tributaries of the Po may be considered as divided into two distinct classes (depending on their origin), namely, Alpine and Apennine. The most important Alpine tributaries are the Ticino, Adda, Oglio, and Mincio Rivers, which flow through Lakes Maggiore, di Lomo, Iseo, and Garda, respectively. This common characteristic of these rivers to form lakes provides a natural settling basin and the tributaries thus deposit all sediment carried in suspension at these places. Below the lakes the rivers are quite clear and carry only some fine sand and mud, so that the Po receives no sediment from these rivers. The clear water tends, rather, to clean out the bed of the river, even removing some of the sediment brought down from the Apennines. The water-sheds that drain into the Apennine tributaries are almost entirely composed of calcites and argillaceous materials and, therefore, the streams are very turbid. In a manner similar to the Alpine tributaries, the Apennine rivers deposit the heavier material in so-called "expansion beds" which extend up stream from the Bologna-Milan Highway for 50 or 60 km. These beds have a variable width of from 1 km. to about 100 m. and a slope not greater than 5 to 8 per cent. After the heavier material has settled out, the rivers flow in eroded beds with a sand bottom and hence carry nothing to the Po except fine materials which are easily carried away by floods, or, as previously stated, by the clear waters of the Alpine tributaries.

There is no sign that indicates, historically, a noticeable advance of the gravels and none that marks any appreciable rise in the river beds, because the floods that occur in these rivers are of short duration and are limited in general to the spring and autumn seasons of the year.

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* Discussion of the paper by John R. Freeman, Past-President, Am. Soc. C. E., continued from September, 1928, *Proceedings*.

† Chf. Insp., Civ. Eng. Dept., Po River, Parma, Italy.

‡ Received by the Secretary, August 15, 1928.

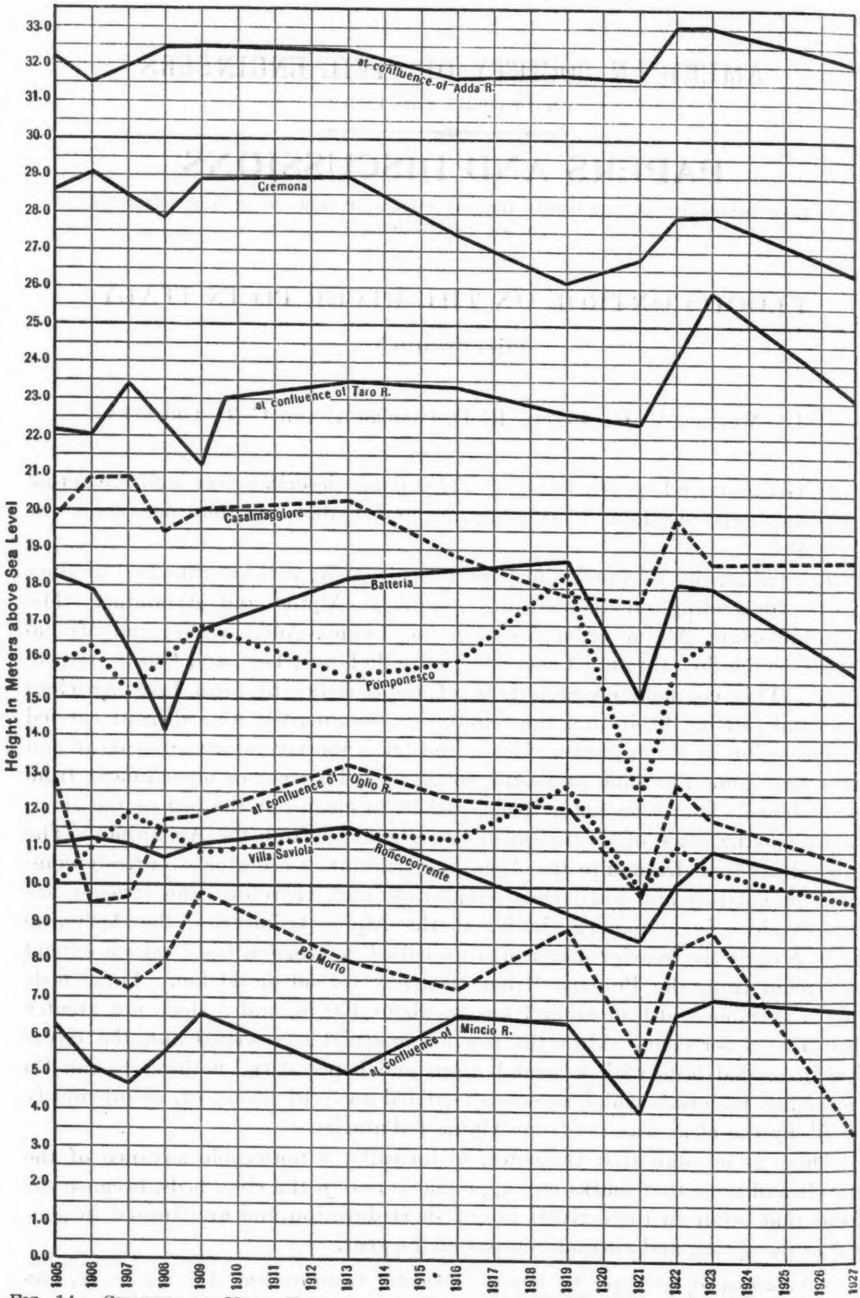

FIG. 14.—CHANGES IN MEAN ELEVATION OF BED OF RIVER PO AT GAUGING STATIONS FROM 1905 TO 1927.

At any rate, the gravel that settles in the expansion beds is heavy enough so that the present slopes may be considerably increased before any of it will be picked up and carried into the eroded river beds below. In the meantime, Italian engineers are providing against such a contingency by reforestation, etc.

This is true also for that part of the Po from the mouth of the Adda River (at Cremona) to the outlet at the sea. In the section from Turin to Cremona several torrents like the Trebbia, the Scrivia, Tanaro, Sesia, Dora Riparia, Dora Baltea, and the Orco Rivers transport gravel to the Po. In the section extending down to Ticino, the bed of the main river is entirely gravel, and the river flow is of a torrential character. The remedy consists in controlling the torrents that flow into the Po and regulating their delta cones.

The slope of the valley on either side of the Po is only 1 or 2% near Cremona. Farther down the river the water level, during ordinary floods, is nearly always above the surrounding country. In this region the adjacent lands are drained by pumping or by means of separate canals that empty directly into the sea.

Lands adjacent to some of the tributaries, such as the Taro, Parma, Enza, Secchia, and Panaro Rivers, are drained by means of pumps.

There is not a second line of dikes along the Po. The Government's dikes form a continuous line of defense down to the sea. However, the lands that are included between these dikes and the lower course of the river are protected by private or communal secondary dikes. These are all lower than the main dikes by at least a meter and, therefore, they are from 20 to 30 cm. below the maximum flood.

In regard to the permanence of the bed of the River Po, Fig. 14 shows the mean elevation of the bed observed at gauging stations for a period of years. The readings are taken at the same place each year. While they are never the same from year to year the elevations always oscillate about a line of equilibrium and show practically no appreciable net change.

This permanence of the average river depth is also clearly demonstrated by other hydraulic phenomena* and experimental results, as follows:

(*a*) A comparison of cross-sections taken during 1916 and 1917 with those observed by the Brioschi Commission, about 1878, show that the area of discharge has not changed. In certain cases, it has increased.

(*b*) The average decennial scalar diagrams of the minimum annual depths at Pontelagoscuro, revised to date by Professor Fantoli,† show an undulating course with alternate minima and maxima, and do not indicate lowering or raising the level of the river bottom.

R. D. Goodrich,‡ M. Am. Soc. C. E. (by letter).§—The author has given the profession a very valuable résumé of the history and development of the regulation of the River Po. To those who have become interested in this particular branch of engineering, the records of the observations, investiga-

* Relazione ed Allegati Minestero dei Lavori Publici, Parma, 1924, p. 84.

† "The Po in the Ephemerides of a Century."

‡ Prof., Civ. and Irrig. Eng., Univ. of Wyoming, Laramie, Wyo.

§ Received by the Secretary, August 25, 1928.

tions, and experience of the eminent engineers who have been engaged on this work in Italy, are not of passing interest only, but are invaluable as sources of information and reference. Hence the paper is of unusual interest and is especially timely on account of the situation connected with the Mississippi River.

While the science of river engineering doubtless had its birth in Italy some centuries ago, the art has been practiced in some of the countries of the Far East for more than 2 000 years. No where is this more true than in the Provinces of Shantung and Chihli, in North China. Some comparisons and contrasts of the practice and conditions there with those in Italy, as described in the paper, will be of value.

One of the largest rivers of the Province of Chihli is the Yung Ting Ho which presents one of the most difficult and interesting problems of regulation and flood control in North China. The single exception is the Yellow River which is, of course, a much larger and longer stream with many unique conditions introducing many special problems.

In 1918 a commission of somewhat international character, was organized by the Chinese Government to carry out some much needed flood-protection and regulation works on the rivers of Chihli, and to survey the general situation and recommend measures for flood relief and the control of its rivers. This Commission therefore has been studying this river, with others, for a short time, although it has been more or less under human control for at least 500 years.

It is about 80 miles from the foothills just below the canyon by which the Yung Ting Ho leaves the mountains, to the confluence with the Hai Ho at Tientsin, 30 miles from the present sea coast. Practically no drainage enters the river below the mountains as this portion has been controlled for a long time by a system of dikes. Above the lowest gorge on the river, the effective drainage area is about 18 000 sq. miles. The basin is approximately 200 miles long and about one-half as wide, although the shape is irregular. The average annual precipitation at Tientsin near the coast, is about 20 in., and at Peking just below the mountains, it is 24 in. However, on the plateaus and mountains comprising the drainage area of this river, the rainfall is very much less than on the coastal plains, but the data are not sufficient for an accurate estimate of the average for the basin as a whole. For this river, therefore, the general conditions as to rainfall on the coastal plains and mountain areas are exactly the reverse of those on the Po, while the amount is probably only about one-third that in Northern Italy.

Through the gorge, the river has a slope of about 1 : 300. Above, it meanders across the Huai Lai Plain for a distance of about 15 miles on a slope of about 1 : 1 000. This plain is at an elevation of about 720 ft. above sea level. To the north and west, above this plain, the river drains a series of mountain ranges and high plateaus.

Summer is the flood season in North China, with the heaviest storms between July 10 and August 20. However, the disastrous effects of the worst storms may not occur near the coast until the waters which accumulate behind the levees on the plains reach their maximum height.

Under present conditions this may not happen until late in September, as in 1917. These great floods, which inundate thousands of square miles of agricultural land, are caused by a succession of typhonic storms. That of July 15 to 17, 1924, produced a maximum discharge of about 176 000 cu. ft. per sec. below the foothills, or about seven-tenths the maximum rate for the Po, in 1917. This is an average run-off for the Yung Ting Ho of almost 10 sec-ft. per sq. mile for the effective drainage area, which may be compared with the 15 sec-ft. stated* for the Italian stream. Mr. Freeman's "possible" explanation for the relatively small run-off per square mile, would seem to apply with even greater force to the Yung Ting Ho where such a large part of the drainage basin is of loess formation, and where there is never any frost in the ground at the time of such storms. The total run-off from this basin for the months of July and August, 1924, was estimated at 53 300 000 000 cu. ft., while the precipitation was approximately 459 000 000 000 cu. ft. This is a run-off of about 11½% for the two months of the flood season.

While these comparisons show that the Po is the larger stream with greater unit run-off and maximum discharge, the discharge of silt by the Yung Ting Ho is much greater. This is perhaps more remarkable than any other feature of these floods of North China, and particularly so for this river. Samples have been taken which showed 10% of silt by weight for the Yung Ting Ho and as high as 11% for the Yellow River. These are of course maximum records, and it is quite probable that such high degrees of saturation are not usually of long duration. Hence, they would have relatively smaller effect on the total weight of silt transported annually; but amounts of from 2 to 5% by weight are not at all uncommon for much longer periods during the flood season. At the same time conditions often arise which tend toward high degrees of saturation locally; for example, when there is rapid erosion of high banks or loess cliffs, which cave off into swift currents so that the stream carries a very heavy load of silt for a short distance. The writer saw a whole village destroyed in three days in 1924 when the Pei Ho was in flood. Deep scouring undercut the banks so rapidly and the shifting of the channel was so inevitable, that the villagers were powerless to prevent the disaster. In fact, the channel has shifted about a mile from east to west within three years at this point.

Such conditions, where the river has built up the plain mostly of this loess formation, and where enormous quantities of similar material are transported annually, probably present the most important factors tending to cause the river to meander by distinct and widely separated channels in this region. The migrations of the Yellow River have been mapped and discussed at various times, and the Yung Ting Ho has had a very similar life history during almost 2 000 years of record.

The average annual volume of silt deposited by the Yung Ting Ho on its way from the mountains to the sea, has been estimated at 13 000 000 cu. yd., in addition to which an unknown volume has been carried on and added to the yellow waters of the Gulf of Chihli to be spread finally on the bed of the sea or along the constantly extended shore. Approximately, this would be 14 000 000 metric tons of material used annually in building up the flood-plain

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 961.

and bed of this river. While the data on which this estimate is based are not conclusive, this subject has been given very careful study and the figures are believed to be conservative. This mass of silt is more than double that brought down by the Po with its larger basin, and yet, be it remembered, does not include that carried to sea. If this material were distributed uniformly over the entire effective drainage area of this river, it would amount to about 850 short tons per sq. mile, which is moved annually from the upper reaches of the river to the plains between the mountains and the sea. While the total amount is certainly much greater than that stated, it should also be remembered that the great bulk of this material comes from certain well-defined areas forming enormous deposits of loess. Much might perhaps be accomplished if this silt problem were attacked at its source, but as yet little or nothing has been done.

For a few miles below the mouth of the gorge where the river leaves the mountains, the bed is composed of very coarse gravel. Less than ten miles from the foot of the mountains the bed is composed almost entirely of sand. Farther down, this material becomes finer and finer, until there is nothing but silt in evidence on the more level plains. Unlike most rivers, the Yung Ting Ho does not now have any well-defined delta cone at the point where it leaves the mountains, due to the fact that it has been trained for centuries between well-maintained dikes, some having been built about 1350 A. D. Although this river now flows into the Pei Ho at a point 30 or 40 miles from the sea, it does have a "delta", so-called from the manner of its formation. This "delta" is from 6 to 20 ft. above the level of the surrounding plain. At its head are at least three distinct lines of dikes constructed to confine the river as it built up its flood-plain and bed, and this building is still going on, as it has for 200 years. Contrary to the opinion of the Italian authority, Frisi, the point where the building up of the bed is most rapid, is 40 or 50 miles below the last sign of gravel in the bed of the stream. Where the river has built up its bed well above the plain the sand and silt forming the river channel is extremely porous, causing enormous seepage losses. In June, 1924, a month before one of the highest floods of recent years, the river was dry at the head of the delta.

It is evident that with a reduction of 100% in the volume of flow, all silt must be deposited, and with a reduction of 50%, which is not uncommon, the amount deposited must be very large, with a decided tendency to maintain the degree of saturation, if it does not actually increase as one proceeds down stream. If the mouth of the Yung Ting Ho is taken at its junction with the Pei Ho on the east side of the delta, the change in elevation at this point in all probability has been little more than it would have been were it building out the coast line at its mouth for the same length of time. The size of this river, the volume and character of the silt transported, the topography and geological formation of its drainage basin, as well as the climatic conditions, are all so different from those obtaining on the Po, that different results and conclusions would seem to be almost inevitable.

In addition to the dikes along the Yung Ting Ho, other devices have been used to assist in the control and regulation of floods. The simplest structure is an open spillway, or notch, in one of the dikes, with abutments and wing-

walls which are usually built of excellent ashlar masonry laid with lime mortar. The crests and aprons are usually a mixture of earth and lime, which forms a sort of concrete. Under certain conditions, and with properly selected materials and workmanship, this method of construction gives very satisfactory results. In other instances abutments and wings as well as the aprons and crest, are of earth-lime concrete, and have stood for almost 200 years. A temporary dike is usually built on the crest of a spillway to a lower elevation than the main dike so that it will be over-topped or can be cut in case of an excessive flood.

Another spillway of cut-stone masonry, is in excellent condition after nearly two hundred years of service. It was intended to be controlled by stop-logs, but the crest elevation is such that only the higher floods overflow it and at present no attempt is made at regulation.

Still another spillway was built for better control of floods on the Yung Ting Ho, as late as 1914, but this one shows foreign (probably French) influence in design and construction. It is located at the west end of the so-called Marco Polo Bridge near Peking, a most interesting and very ancient structure mentioned by that noted traveler. Between these last-mentioned weirs, the river dikes are very wide apart and form a channel reservoir having an area of more than 20 sq. miles, which has great effect in reducing flood peaks on account of the relatively large storage effect and also on account of the very large absorption which takes place.

The earth dikes have piles of material distributed along the top for emergency repairs. This is common practice although the earth available is usually not sufficient for more than very minor repairs. The elevation of the channel reservoir above the surrounding country, together with the very porous nature of the deposits made by the river along its flood channel, doubtless accounts for the extraordinary seepage losses along this reach of the river. It is estimated that for a flood peak of 5 000 cu. m. above Lu Kou Chiao, about 1 100 cu. m. will be discharged over the weir at that place, while about 450 cu. m. more will flow over the cut-stone spillway. Between these points there is a probable further reduction in the crest discharge of about 700 cu. m. for the reasons stated.

All this may serve to emphasize the well-known facts that, while observation and experience with one stream are valuable in that the statement and analysis of the problems involved on that stream may be recorded for comparison, each river presents its own peculiar problems due to its own peculiar conditions. Only by the most careful study of the conditions and problems for each individual case, can one hope to arrive at anything approaching a satisfactory solution.

C. E. Grunsky,* Past-President, Am. Soc. C. E. (by letter).†—Current engineering literature is filled with accounts of works and structures recently completed and with discussions of pending problems. It is refreshing to get from time to time what Mr. Freeman has given the profession—an

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† Received by the Secretary, September 24, 1928.

account* of works which have functioned for centuries and which experience has shown to be adapted to the purpose for which they were constructed.

In the matter of flood control, the Engineering Profession has much to learn from this experience. There is only one point in connection therewith which, it seems to the writer, should be further stressed. The valley of the Po differs from the valley of the Sacramento and of the Mississippi mainly in the fact that the principal flood-control levees can be set far back from the stream in the upper parts of the valley. Broad areas of cultivable land protected against moderately high river stages by low submersible levees—summer dikes—flank both sides of the river. These have the same purpose and function in the same way at extreme river stages as the by-pass areas in the Sacramento Valley. In the case of the Sacramento Valley, however, the areas set apart for occasional flooding could not, for topographical reasons, be located along the river; they had to be set back some distance from the high bank land. In both cases, however, the purpose is the same, that is, large cross-sectional area is provided in or along the river's upper reaches to hold water in temporary storage, thereby cutting down the maximum rate of discharge at points in the valley farther down stream.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 957.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

ADMINISTRATIVE WATER PROBLEMS
A SYMPOSIUM
Discussion*

By Thomas R. Newell, Assoc. M. Am. Soc. C. E.

Thomas R. Newell,† Assoc. M. Am. Soc. C. E. (by letter).‡—An outline of the methods used in transmitting and delivering reservoir water in Water District No. 36, Idaho, has been presented by Mr. Baldwin. It is mentioned§ that transmission is effected through the Snake River channel from both the Jackson Lake and American Falls Reservoirs to the river head-gates of the numerous owners of storage rights. Thus, the channel of the main river carries an increment of reservoir in addition to the current run-off of the water-shed during a large part of the irrigation season. Natural water distribution must be made simultaneously with storage delivery despite the complications introduced by mingling this current run-off with these continuous streams of transit storage. In some sections of the river the reservoir water increment is even greater than the natural flow during part of the season. The relative proportions vary for years of differing snow supplies and irrigation demands.

Transmission loss schedules and reservoir outlet divisions always must meet the tests of changing conditions and even of changing ownerships. The small percentage of ultimate wastage (flow past Milner Dam during regulation period§) is a real index of the excellence and efficiency of the control system of river operation as a whole. The fact that these major reservoirs have been operated ever since their construction in conjunction with natural water distribution from the same water-shed, through identical delivery channels and without the authority of a Court Commissioner, speaks well for

* Discussion on the Symposium on Administrative Water Problems continued from September, 1928, *Proceedings*.

† Idaho Falls, Idaho.

‡ Received by the Secretary, September 17, 1928.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1081.

the efforts of the administrative official and perhaps also for the tolerance of district water users.

The compromise agreements upon which division is made between natural and reservoir waters when flowing commingled are of particular interest to the writer. In Water District No. 36 these agreements consist of brief digests of methods of division at reservoir outlets and schedules of percentage transmission losses to be applied through successive river sections to transit storage. As they are adopted annually in slightly revised form, these compromise schedules should carry greater prestige from year to year, and as time goes on, should become more nearly representative of true conditions. It is readily seen that variations in these schedules directly affect the quantity of reservoir delivery. In like manner, changes affect the total supply of natural water in the opposite direction. Even now basic changes are stubbornly contested and are almost impossible of adoption without recourse to hydrological investigations of a co-operative nature. Objections of merit are usually met by secondary arrangements or the granting of special privileges to avoid upsetting standard compromise schedules.

In this connection it is interesting to note the exact effect of a change in a river operation agreement from the viewpoint of a holder of a natural water decree and a user of reservoir water. Assume, for example, that a transmission loss schedule is increased 10 per cent. Each storage owner's holdings are decreased 10 per cent. The only escape he has from the penalty is that his storage right may be a mere insurance and seldom withdrawn. This same increase in applied transmission loss increases the total supply of natural water, but not *pro rata* to each holder of a decreed right. In Idaho, a decreed right specifies a quantity (if beneficially used) and, in all cases, a priority date. As river supplies decrease below the quantity necessary to fulfill all decrees in use, the river water-master refers to his chronological list of priority dates and declares that right with the latest date either partly or completely invalid and denies use thereunder. This process of eliminating one user after another is continued until the river stage reaches the summer low. Then, as supplies increase, each decree is restored in reverse order.

Pioneer rights to the extent of the minimum summer flow remain untouched and, in reality, have no interest in the changed transmission loss. The younger the right the longer is the period of denied use which must be suffered. To each invalidated right an increased transmission loss means an extended time of validity at either end of its period of denied use. The length of the extension is defined by the amount of gain to natural supplies due to the changed schedule and the rate of river decline or increase. It may not encompass the whole decree and it may not help for more than a single day; but for several years the storage delivery rate from the Jackson Lake Reservoir has been quite large during the months of July and August. Likewise, the summer minimum of Snake River flow usually remains fairly constant for several weeks in the height of the irrigation season. Therefore, it is possible for a single large decree, properly placed along the chronological scale, to garner a lion's share of the total increase in natural supplies resulting from a changed transmission loss.

To carry the illustration still further, it is true that many of these young rights have purchased supplemental storage, so that their gains may be partly offset by losses to their supplemental supplies. Careful study of these effects serves to accent the fallacy of the popular belief that any general method of division between the two classes which balances for the season is fair to the individual decree holder. It may also present to the politically minded an interesting angle of the "ins and outs" of stipulative agreements.

Compromise river and reservoir agreements should tend toward workable methods of division which should be fair to each party throughout each season and which should keep pace with current events. With the initial draft from the new American Falls Reservoir comes a series of changed ownerships, new river complications, and notable variations in river regimen extending from Jackson Lake all the way to Milner. A reservoir outlet division, operated on the contour method, in which evaporation losses and natural storage credits are balanced against bank storage return, usually contemplates a full use of the reservoir and cannot be expected to serve a hold-over storage unit. Henrys Lake Reservoir is notably in the latter class and on account of recent changes in ownership at Jackson Lake, this great storage unit is tending in that direction.

These same factors operate toward less regular increments of reservoir water being carried through the main river channel. Flat percentage rates of transmission loss and flat time interval allowances for any and all river stages and rates of delivery will not so fairly comprehend the new situation. Lag credits help to even the charges, but should be applied in direct proportion to increases in stage and should not be permitted to be used in any but a regular mechanical manner. The space privilege, the temporary transfer, and many permanent transfers might each serve an honest purpose did they not so often go hand in hand with appropriation and even Court decree beyond the bounds of beneficial use. The current dearth of water supply in Water District No. 36, due to the many new storage purchases (made possible by the construction of the American Falls Reservoir, and the ample natural run-off of 1927 and 1928), has eased the river operation situation temporarily. The advent of a season of shortage will not soon bring general crop losses on account of the presence in the district reservoirs of much "water insurance" which may be rented; but with the reclamation of additional lands, now contemplated, supplies will eventually be depleted to the point where river and reservoir problems will become more important.

Only a part of the transmission and division agreements now in force have support in actual seepage investigations. Of these investigations, many have been continued during one season only, while some are based only on miscellaneous field runs. Continuity through different types of seasons—the fundamental of every thorough hydrological study—has been denied in most cases. Few, if any of the channel studies made in the past, show the application of the modern methods of refining river-station differences in use in California and some even ignore surface tributary inflow as an independent variable. The Kootenai River study now in progress on the International Boundary has been planned with a view to obtaining a degree of refinement

far beyond that of any lake or reservoir segregation yet made in this district. The criticism often heard, to the effect that the segregation of the many factors involved can never be exact, is true despite the use of the most modern methods economically feasible.

However, real values do result from orderly presentation of measurable factors properly qualified as to exactitude. Just divisions require careful observation of seasonal trends of disputed factors. Developed waters resulting from drainage recoveries, return flows from new irrigation, artificial ground storage, and reservoir bank storage, should each be observed currently. Changed plans for reservoir drafts due to changing ownerships should not be overlooked. All in all, too much should not be expected of old and insufficient investigations, and every effort should be made to make the new ones more thorough and continuous. The effort to centralize and co-ordinate all administrative activities, within the district, has built up a continuing and an adequate organization which has effected improved operation service. The basic schedules underlying operation schedules are entitled to equal consideration and support.

To the student of river operation the paper is particularly interesting, in that it is representative of ten years' experience in transmitting reservoir waters at rates of much greater magnitude than the usual. The same rules which apply to storage deliveries at the rate of a few miner's inches for a few miles are entirely inadequate in transmitting reservoir water by the thousands of second-feet for hundreds of miles to the respective owners. It is to be regretted that more detail has not been permitted for a careful description of the very effective system of control which has resulted in uninterrupted service and, at the same time, a negligible ultimate waste. The attainment of these two ends is the mark of good regulation and deserves the careful consideration of every person interested in orderly distribution on a large scale.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

TRANS-MOUNTAIN WATER DIVERSIONS

A SYMPOSIUM

Discussion*

By Messrs. B. F. Jakobsen and Paul M. Entenman

B. F. Jakobsen,[†] M. Am. Soc. C. E. (by letter).[‡]—Mr. Van Norman states[§] that a careful investigation shows that Los Angeles will reach the limit of its water supply within ten years, but he does not give any authority for this statement. Careful engineering investigations made by competent engineers, such as Thomas H. Means, M. Am. Soc. C. E., Paul Bailey, M. Am. Soc. C. E., and others, show that additional water supply can be obtained from Owens Valley and Mono Basin in sufficient quantity to justify the building of another aqueduct from Owens Valley with a capacity of about 450 sec-ft., and that the power which can be developed would pay for this additional aqueduct. This water is ideal in quality for domestic use, and would supply a population of about 4 000 000. It will most likely be much more than "ten years" before this population is reached.

The author refers[‖] to a large infiltration canal. There will be about 8 000 acre-ft. of mud to be removed each year from the 1 500 sec-ft. of flow, for which Los Angeles has applied from the Colorado River. Assume the width of the available river area to be 528 ft., or 0.1 mile, or an area of 64 acres per mile length of infiltration canal. If the silt is deposited in the filter to a depth of 2 ft. and there are 25% voids, 250 miles of infiltration canal per year will be needed. This does not look very practical, and the writer doubts whether the usual methods of mechanical filtration, which the author suggests as an alternative, will work much better.

The storage capacity of Bridge Canyon Gravity Aqueduct diversion is given by Mr. Van Norman as 6 200 000 acre-ft.,[¶] and he states that this

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* Discussion on the Symposium on Trans-Mountain Water Diversions, continued from September, 1928, *Proceedings*.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, July 23, 1928.

§ *Proceedings*, Am. Soc. C. E., Papers and Discussions, April, 1928, p. 1116.

‖ *Loc. cit.*, p. 1118.

¶ *Loc. cit.*, p. 1119.

would be filled with silt before many years, and that, therefore, if this site were selected, it would require an additional dam within a short time. At this point the river carries about 100 000 acre-ft. of silt annually, so that if all the silt were deposited, it would still take sixty-two years to fill the reservoir. Long before the silt problem would become serious, other dams would have been built in the natural course of developing the river.

Mr. Van Norman states* that the two gravity routes have been investigated and found "extremely expensive" to construct, and that the time element needed for construction make them impracticable. In a recent article the author stated† that the annual pumping cost of the proposed project will be $8 400 000. Capitalizing this at 4.5% per year gives $187 000 000. If Mr. Van Norman will add this to the cost of the proposed pumping plan, the writer believes he may find it difficult to maintain his assertion that the gravity routes will be "extremely expensive."

Doubt has been expressed by the author whether a dam higher than 630 ft. above the low-water stage of the river can be safely built at the Bridge Canyon site.* A geological report described the rock above the exposed igneous rock as hard quartzitic sandstone. It might be well to leave this an open question until decision can be made on the most competent geological and engineering advice.

Much value would be added to this paper if Mr. Van Norman would give the detailed cost data which he states‡ have been prepared. It would be especially interesting to compare the two proposed gravity routes with the proposed pumping scheme with its enormous annual pumping charges.

Mr. Van Norman states§ that the most favorable location for a power dam is at Boulder Canyon. A number of such sites have been described in a report‖ by E. C. La Rue, M. Am. Soc. C. E., according to which it would be a serious mistake to construct the proposed Boulder Canyon Dam, since it does not fit into a general plan of development of the river, and would result in a waste of about 300 000 h.p. William Kelly, M. Am. Soc. C. E., while Chief Engineer for the Federal Power Commission, also investigated the proposed Boulder Canyon Dam and recommended against it.

The author has failed to mention the opinions and ideas of competent engineers who differ with him. Some plan should be arranged which will receive the approval of a considerable number of competent engineers, and it should be understood that there is nothing final about the proposed plan.

PAUL M. ENTENMAN,¶ ASSOC. M. AM. SOC. C. E. (by letter).**—Mr. Van Norman's paper shows clearly that, of the routes surveyed for the Los Angeles-Colorado River Aqueduct, there are but two that deserve serious consideration —the Blythe route and the Black Canyon route. As the Black Canyon is pred-

* *Proceedings*, Am. Soc. C. E., Papers and Discussions, April, 1928.

† *Engineering News-Record*, May 31, 1928, p. 853.

‡ *Loc. cit.*, p. 1121.

§ *Loc. cit.*, p. 1122.

‖ Report No. 556, U. S. Geological Survey.

¶ Cons. Engr., Banning, Calif.

** Received by the Secretary, July 23, 1928.

icated on the choice of a site for the construction of the proposed Boulder Dam (which may not be the one finally selected) the adoption of the Blythe route, or a route farther south, seems inevitable. This is apparent since it will be desirable to begin early construction of the aqueduct, whereas the approval of legislation authorizing the Boulder Canyon project may be delayed.

The author states* that investigations of the feasibility of the Blythe route as a project independent of the construction of the Boulder Dam, are being conducted. Without a high dam creating a reservoir that will act as a desilting basin, the problem of excluding the silt of the Colorado River from the aqueduct will be one of major importance.

The proposed intake is located 120 miles below the canyon, in the section of the river characterized by broad, fertile, deltaic plains. In this region, the river flows in a bed of its own alluvium. Logs of wells and other subsurface explorations show that the alluvial deposit is of more or less uniform character for great depths. It is marked by extreme fineness of the constituent particles. Underground flow through this material is so slow that it is doubful whether an infiltration basin of sufficient capacity to serve the aqueduct would be feasible, for irrespective of the porosity of the material in which the infiltration canal may be constructed, flow into the canal must be through the fine bed silts of the river.

The rate of underground flow, investigated with Hazen's well-known formula,† assuming $d = 0.10$ mm., is found to be a velocity too low to serve an infiltration canal of reasonable dimensions.

The assumed value of $d = 0.10$ mm. is larger than the effective size of the sand grains in the bed of the Colorado. Fortier and Blaney‡ give a mechanical analysis of the deposits in the bed of the river from a number of samples; one, taken at the intake of the Palo Verde Canal, within five miles of the site of the proposed intake for the Los Angeles Aqueduct, shows that 14.62% of the sample consisted of grains having a diameter of 0.05 mm.§

A theory that extensive strata of water-bearing gravels may be found below the silt and sand has considerable currency. The investigations of Fortier and Blaney show clearly that very little material that can be classified as coarse sand reaches the lower river.‖ The canyon section of the stream is an efficient grinding mill reducing boulders, gravel, and sand to a silt of which more than 50% is finer than Portland cement.

Occasional beds of gravel are found where débris cones at the mouths of washes debouch directly into the river; but even then gravel is not found in extended sheets. During floods, the river has sufficient velocity to transport any gravel or small boulders which may enter it from a wash; and successive floods cause a wide distribution of this débris. Such gravel beds as are found in the Colorado River occur in lenses or in long trains of coarse material in the buried channel of the stream.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1118.

† "Filtration of Public Water Supplies," by Allen Hazen, M. Am. Soc. C. E.

‡ "Silt in the Colorado River," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 67*, U. S. Bureau of Public Roads.

§ *Loc. cit.*, p. 73, Table 51.

‖ *Loc. cit.*, p. 9.

As it is unlikely that an infiltration canal of sufficient capacity to serve the proposed Los Angeles Aqueduct will be feasible, the intake structure must include some desilting device. The most successful desilting structure on the Lower Colorado is the settling channel and sluice-way at Laguna Dam. A diversion weir raises the level of the river about 10 ft. The settling basin is 650 ft. long, with a cross-section 126 ft. wide at the top, 116 ft. at the bottom, and 18 ft. deep. At its down-stream end are three sluice-way openings, 33 ft. wide, controlled by Stoney roller gates. Water is skimmed from the surface of the settling channel through a gate with thirty-five 7½-ft. openings regulated by flash-boards.

Once a week, the sluice-gates are opened, and the silt which has accumulated in the settling channel is sluiced back into the river below the Laguna Dam. The settling channel is effective in removing all the bed silt and about 50% of the suspended silt in the river.*

The site of the proposed intake for the Los Angeles Aqueduct is not favorable for the construction of a weir, but as all the water has to be pumped in any event, head for sluicing purposes could be developed with pumps. However, unless the fluctuations in discharge are to be controlled completely by the construction of the Boulder Canyon Reservoir, some form of diversion structure will be necessary. The experience of the Imperial Irrigation District with rock-fill dams across the Colorado shows that a much cheaper structure than the Laguna Dam, such as that closing the Bee River channel,† will serve this purpose, provided there is ample spillway capacity to prevent overtopping.

The settling channel for the proposed aqueduct should be designed to reduce the velocity in this basin to less than ½ ft. per sec.; and the gates controlling the flow to the aqueduct should be of such capacity that only the surface water between 6 in. and 1 ft. in depth will be skimmed from the settling channel. Some device other than flash-boards should provide regulation of flow. It is surely not beyond the ingenuity of engineers to design a skimming gate that will be tighter than a flash-board structure. At Laguna Dam, it has been found that some of the silt which enters the canal leaks through and around the flash-boards, and that more silt than is actually suspended in the surface water of the settling basin flows into the canal through the boards and by reason of vertical currents that form along the face of the regulating gates. Laboratory studies of the control of these vertical currents and of the design of a form of hinged gate that would replace the crude flash-boards is warranted for a project of the magnitude of the Los Angeles Aqueduct.

The quantity of silt carried by the Lower Colorado River varies with the discharge. At the lower end of the canyons 0.86%, by volume, is an average. When the aqueduct is operated to full capacity, this would mean transporting nearly 10 000 acre-ft. of silt each year. Unless this silt is removed at the intake, its deposition in storage reservoirs along the route of the aqueduct would rapidly diminish their capacity.

* *Technical Bulletin No. 67*, U. S. Bureau of Public Roads, p. 89.

† "A River Diversion on the Delta of the Colorado," by S. L. Rothery, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412.

The silt should be removed at the river for another reason. Its abrasive nature is destructive of the working parts of pumps. More than 70% is silica, and its exceeding fineness permits its penetrating close spaces. An ordinary centrifugal pump will have its bearings ground out in a month's or six weeks' operation. Pumps are made with outboard bearings to meet this problem, but the runners and shell wear rapidly, especially when operated at high speeds and against high heads.

If the head for sluicing purposes is developed with pumps instead of with a diversion dam, such pumps would have to handle silty water. However, the lift would be comparatively low, and large-capacity, low-lift pumps of the propeller type have handled Colorado River water without excessive wear. That settling channels are completely successful for the removal of silt has been demonstrated on a small scale in the water-works plant at Calexico, Calif.

Any study of the silt problem of the Colorado as it affects a domestic supply for Los Angeles, Calif., as well as the irrigation projects in the lower river, emphasizes the importance of constructing some desilting structure in the canyons of the river. The most recent computations of the quantity of silt transported to the end of the canyon section are those of Fortier and Blaney who estimate that 137 000 acre-ft. reach the lower river annually.* They conclude that: "The most feasible end economical means of solving the silt problem * * * is to impound the river silt behind a high dam, such as is proposed at Boulder Canyon."

* *Technical Bulletin No. 67,* U. S. Bureau of Public Roads, p. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MODERNIZING THE SUBURBAN TRANSIT OF THE METROPOLITAN DISTRICT

Discussion*

BY MESSRS. JOHN A. MILLER, JR., DANIEL L. TURNER, BILLINGS WILSON, FRED LAVIS, GUSTAV LINDENTHAL, AND CHARLES EVAN FOWLER

JOHN A. MILLER, JR.,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The problem of suburban transit in the Metropolitan District of New York City is twofold. It involves, first, the transportation of residents of suburban communities to Manhattan Island; and, second, their distribution after arrival. At present, these functions are being performed by separate agencies. The railroads bring their passengers to terminals on or close to Manhattan Island, and the local transportation agencies distribute them to their ultimate destinations. Proposals have been made from time to time that both functions should be performed by the suburban transit agencies by extending their lines through the heart of Manhattan, thus relieving the local transit facilities of this burden. A plan of this kind is suggested by the author. Undoubtedly, this plan has a number of advantages, but it has also some decided disadvantages, particularly as applied to the suburban district of Northern New Jersey.

The scattered character of the residential districts in North Jersey and the diversity of locations of places of business in New York City makes it practically impossible to devise a system whereby one can make the trip from one's home to one's place of business in a single vehicle. The existing suburban steam railroad lines in New Jersey have seventeen distinct branches, requiring individual train service. On account of the length of these lines each of the seventeen now has two or three or sometimes four classes of trains—local, express, and semi-express, or trains running express on the

* This discussion (of the paper by Francis Lee Stuart, M. Am. Soc. C. E., published in May, 1928, *Proceedings*, and presented at the meeting of September 5, 1928), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Editor, *Aera*, and Associate Editor, *Electric Railway Journal*, New York, N. Y.

‡ Received by the Secretary, May 23, 1928.

outer end of the line and local nearer the city, or *vice versa.* Assuming that the train service of each of the seventeen lines is of only two classes, express and local, then at least thirty-four different kinds of trains would have to be run over the distributing system on Manhattan Island to give through service to each of the outlying stations on each of the different lines. The difficulties involved in complicated operation of this kind are obvious. Moreover, they are unavoidable in any system that attempts to collect passengers from widely scattered residential districts, transport them to Manhattan, and distribute them over an area 4 miles long and 2 miles wide. It seems unlikely that any transit system can be built both to collect and distribute the traffic between New York City and Northern New Jersey.

Distribution of the New Jersey commuters to their destinations in New York City is a complicated problem in itself. Investigations by the North Jersey Transit Commission show that about 240 000 passengers daily originate, or are delivered by the railroads, at points on the New Jersey side below the line of 42d Street, New York. Of these, approximately 60 000 desire to reach destinations north of 42d Street. Conversely, of 42 000 who originate north of this line on the New Jersey side, nearly 30 000 are bound for points in down-town Manhattan. In general, they cross the river on east and west lines and complete their journeys by using north and south routes in New York City. Fig. 1 shows the volume and direction of passenger movement across the Hudson River in straight lines from point of origin to destination, and Fig. 2 shows the routes actually followed.*

From a study of these charts it is evident that a majority of the New Jersey commuters are following indirect and inconvenient routes to reach their destinations. It is evident also that the paths of desired movement are so diverse, that it would be impossible to design any single distributing agency to accommodate them all.

A study of Fig. 1 showing the direction of passenger movement from point of origin to point of destination, however, suggests a possible solution of the problem. It will be seen that nearly all these lines cross the Hudson River diagonally. Only one type of transportation facility does that, or can do it. This facility is the humble, old-fashioned ferry. Despite a more or less general impression that ferries are obsolete as a means to passenger transportation, there is reason to believe, as the writer stated in a recent article in the New York *Herald-Tribune,* that they offer the best solution of this particular problem.

Paradoxical though it may seem, the building of the Holland Vehicular Tunnel between New York and New Jersey really has created a new opportunity for the Hudson River ferries. In the past these ferries have been operated primarily for the accommodation of vehicular traffic. Passenger carrying has been a secondary consideration. Now, the tunnel has taken away a substantial part of the vehicular traffic from the ferries, and a genuine opportunity exists to remodel the ferry service to provide rapid and convenient passenger transportation.

* Based on data of the North Jersey Transit Commission.

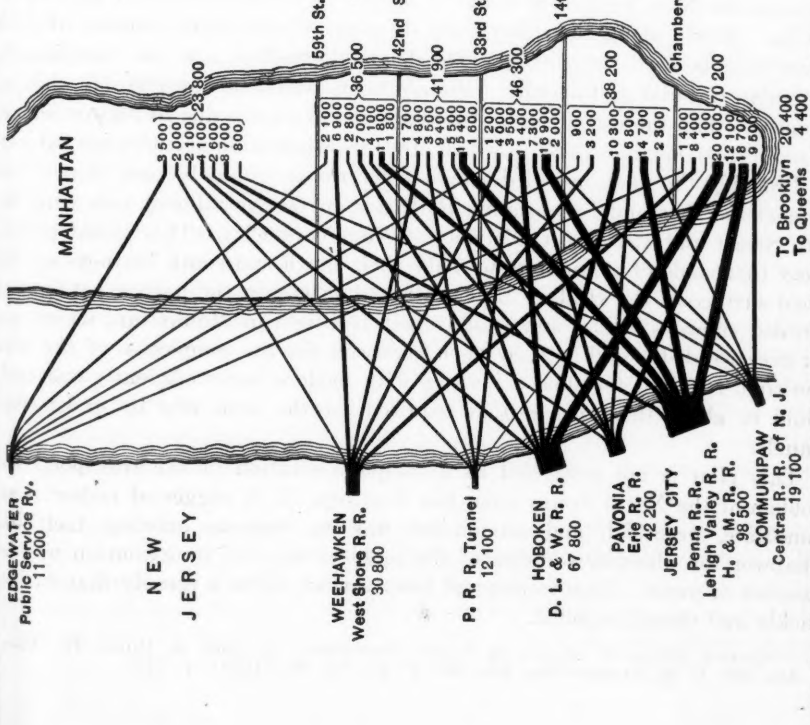

FIG. 1.—DIRECTION OF DESIRED PASSENGER MOVEMENTS IN STRAIGHT LINES FROM NEW JERSEY POINTS TO NEW YORK DESTINATIONS.

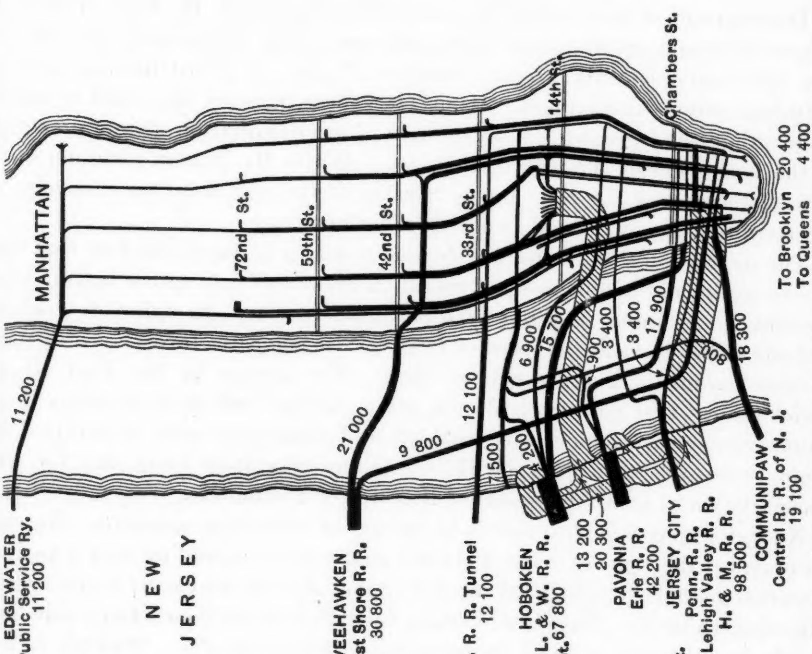

FIG. 2.—PRESENT DAILY MOVEMENT OF PASSENGERS ACROSS THE HUDSON RIVER FROM NEW JERSEY TO NEW YORK.

Development of diagonal ferry routes, following closely the lines of desired movement, would provide more direct and convenient service than now exists. The cost would be only a small fraction of that of a distributing agency including tunnels and bridges. About 4 000 ferry trips per day could be made for less than the mere interest on the cost of the distribution system proposed by the North Jersey Transit Commission. While Mr. Stuart gives no estimate of the cost of the New Jersey Section of the system he recommends, it is certain that it would equal hundreds of millions.

The impression that ferries are obsolete probably is due to the fact that the present equipment and methods of operation are out of date rather than to any inherent shortcoming in water transportation service. In spite of obsolete methods and equipment the speed of ferry service compares favorably with that of other means of making similar trips. The history of the East River ferries* shows that passengers do not prefer bridges and tunnels unless they really are more convenient. If methods and equipment were modernized it would be possible to utilize the Hudson River effectively every day for the transportation of many thousands of New Jersey commuters.

No serious physical obstacles lie in the way of such ferry operation. Nature has provided the right of way. Personal observations extending over a period of several years have convinced the writer that it is by no means overcrowded with other shipping. Sufficient docking facilities on the New Jersey side are already in existence or could be arranged without difficulty. Docking facilities on the New York side would be more expensive, but would be easy to obtain. A trip along the water-front discloses a substantial amount of dock space that is used very little. The boats themselves are not inordinately expensive to build and operate. Modern boats would be considerably cheaper to operate than those now used. Interruptions to service due to fogs or ice are no more frequent than those to which other transportation agencies are subject.

Probably the most useful diagonal ferry routes would be ones that would connect the Erie and Lackawanna Railroads on the New Jersey side with the 34th Street and 42d Street Districts of New York City. This would permit many thousands of commuters to make their north and south journeys on the broad surface of the Hudson River rather than within the narrow confines of crowded subways. East and west transit facilities in Manhattan, which are not overcrowded, could be used when necessary for the completion of the trip. The time required for such a journey with modern boats efficiently operated, would be about the same as that required for the same trip by under-river tunnels.

This plan is not suggested as a complete solution of the transportation problem of the North Jersey suburban territory. It is suggested rather as an immediate, practical, and economical way to improve existing facilities. Whatever the ultimate solution of the problem may be, its execution will be a matter of years. Improvement of ferry service offers a remedy that can be quickly and cheaply applied.

* "Certain Effects of Bridges on Transit Conditions," by John A. Miller, Jr., Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 127.

DANIEL L. TURNER,* M. AM. SOC. C. E. (by letter).†—The author is rendering a great public service in interesting himself in this way in the New York suburban transit problem. The Suburban Transit Engineering Board greatly appreciates the efforts he has made, and is making, in helping to find a proper solution of the problem. Mr. Stuart has appeared before the Board, at its invitation, and has given it the benefit of his large experience and his views, which are being carefully considered at every phase of its studies.

As a member of the Board and from the point of view of that body, it would be premature and ill-advised for the writer to express any personal opinions at this time as to how the New York suburban transit problem should be dealt with. The Board is studying the problem intensively and in due course hopes to present a satisfactory solution, but it has not yet reached any conclusions that can be presented publicly.

As Mr. Stuart has stated,‡ approximately 40% of all the steam railroad passengers carried in the United States originate within about 40 miles of the City Hall in Manhattan. In other words, the yearly railroad passenger traffic to and from the terminals in the New York Metropolitan District in 1926 amounted approximately to 360 000 000 passengers as compared with nearly 900 000 000 passengers carried in the United States during that year. Furthermore, the average journey per passenger on all the railroads in the country in 1926 was about 41 miles, which means that the average steam railroad passenger throughout the United States only rides far enough to carry him from the center to the limits of the New York Metropolitan District. These figures give some idea of the magnitude of the problem, but more important still is the ratio of the commuters to the total railroad passengers in the Metropolitan District.

In 1925, in the New Jersey Sector of the District, it was estimated that about 165 000 000 out of a total of 204 000 000 passengers were commuters; that is, about 81% of the annual passenger traffic on the railroads in the New Jersey Sector were regular riders from home to work and back again day by day and month by month, throughout the year, at something less than the regular one-way fare. In the Westchester Sector 67% of the passengers were commuters and, in the Long Island Sector, 78 per cent. Altogether, approximately 284 000 000 out of the 360 000 000 passengers, or 78% of the total railroad passengers in the District in 1926 were commuters. These last figures show how vital the suburban transit problem is to the steam railroads serving the Metropolitan Area. The proper solution of the problem is essential to their welfare, and the problem cannot be adequately solved without their co-operation.

At the outset the Board recognized that there were three sections in the suburban region around New York City—the Long Island Section, the Westchester Section, and the New Jersey Section—and that the problem is different in each and requires a different local solution. Therefore, the Board is proceeding from the local to the general solution of the problem, thereby recogniz-

* Cons. Engr.; Chairman, Suburban Transit Eng. Board, New York, N. Y.

† Received by the Secretary, September 5, 1928.

‡ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1391.

ing the desirability of securing the best local solution as a primary consideration. Consequently, the local conditions and needs in each section are being studied separately and intensively in order to produce a plan that will be satisfactory to that section. On the other hand, all the sectional plans must have one feature in common; that is, the delivery part of the system in Manhattan. In other words, in addition to the lines traversing the suburban area, each local system must be provided with a subway system which must be constructed under the streets of Manhattan to deliver the city-bound suburban passengers to their city destinations, whether such passengers come from Long Island, Westchester, or New Jersey. After the proper local plans have been developed, they will be co-ordinated at the center as far as practicable into a single unified Manhattan delivery subway system, which will serve all the surrounding suburban sections jointly.

The principal advantages in a unified delivery system in Manhattan are that:

1.—It will afford an easy means for passengers to interchange from the trains of one local system to those of another local system, and thus make all sections of the region conveniently accessible to each other.

2.—It will avoid duplicating the delivery subways in Manhattan for the separate use of each local system, thus reducing the subway mileage and thereby saving construction costs.

3.—And, likewise, it will reduce the cost of operation.

The make-up of the Suburban Board is well adapted to develop a plan in the way described. In the first place, the personnel of the Board not only includes representatives of all the public agencies interested in the suburban transit problem, but it also includes representatives of all the railroads affected. For example, New York City is represented by the Chief Engineer of the Board of Transportation; Westchester County, by its County Engineer; Nassau County, by its County Engineer; Suffolk County, by its County Superintendent of Highways; New Jersey, by the Consulting Engineer of its North Jersey Transit Commission; the Port of New York Authority, by its Deputy Manager; the New York Central, the New York, New Haven and Hartford, and the New York, Ontario and Western Railroads, by the Engineering Assistant to the President of the New York Central Lines; the Erie, the Delaware, Lackawanna and Western, and the Lehigh Valley Railroads, by the Vice-President and Chief Engineer of the Erie System; and the Pennsylvania, the Central of New Jersey, the Long Island, and the Baltimore and Ohio Railroads, by the General Manager of the Eastern Region of the Pennsylvania Railroad. Each of these Board members was officially designated to serve. The fact that these official representatives of all the interested agencies have joined their efforts to work out a solution of the problem is an important feature of the Board. Moreover, this is the first time that the railroads have officially participated in solving the suburban transit problem, which affects them just as vitally as it does the public. In this respect, the Board is unique.

In the second place, the preparation of a plan for each section has been assigned to a committee of the Board composed of those members specifically

charged with representing such section, in order that they may produce plans which will best serve their sections locally. In this way the greatest degree of local independence is secured to each suburban section in developing its own plan. In effect, this means that within the Suburban Board there are really three independent local boards functioning, each dealing with its own suburban problem. To illustrate, take Long Island as an example. There is a Long Island Committee working out the plans for Long Island, consisting of the County Engineer of Nassau, the County Superintendent of Highways of Suffolk, the Chief Engineer of the Board of Transportation, the General Manager of the Eastern Region of the Pennsylvania Railroad, and the Chairman, Vice-Chairman, and Secretary of the Suburban Board as *ex officio* members. Similar committees are developing plans for Westchester and North Jersey. The problem has never been approached in this way before.

As soon as the local committees have developed the plans that will serve their suburban areas best, and will furnish their several groups of passengers with a proper delivery subway system in Manhattan, then the whole Suburban Board, from the regional point of view particularly, will co-ordinate the several subways in Manhattan into a unified system, as already described.

This method of producing a comprehensive suburban transit plan prevents one section from securing advantages at the expense of the others. It reduces to a minimum the danger of developing an unbalanced plan. It means that the problem will be visualized from the standpoint of the needs of each suburban area as well as from that of the entire region; and, finally, it seems to insure a plan that will be well thought out, comprehensive in scope, capable of being harmoniously related to the existing railroads, relatively economical to construct and operate, and adapted to both local and regional suburban transit requirements.

BILLINGS WILSON,* ASSOC. M. AM. SOC. C. E. (by letter).†—This paper is all too brief, but, fortunately, it is very pithy. The author has deftly located and charted all the important high spots in his rapid flight through this transit "fog". It remains for those who follow to "get down to cases" and see if it is practical to construct a right of way along the "trail" he blazes.

By virtue of his connection with the Port of New York Authority, the writer has been giving considerable thought to this particular question in connection with the work of the so-called Suburban Transit Engineering Board. This Board is comprised of representatives of Northern New Jersey, Westchester, Nassau, and Suffolk Counties, New York, the City of New York, all the railroads, and the Port Authority, and was created in response to a legislative request that the Port Authority undertake a study of ways and means for improving suburban transit conditions—more familiarly known as the commuter problem. The Board has been functioning for about a year.

With the cordial co-operation and assistance of all the carriers, it has collected and analyzed very valuable statistics on the commuter traffic which focuses on Manhattan Island. The Board has had, and is having, the

* Deputy Mgr., The Port of New York Authority, New York, N. Y.

† Received by the Secretary, September 8, 1928.

co-operation of transit students, such as the author, to whom it feels deeply indebted.

Suburban transit is here considered as distinct from the urban or intra-city problem. About 500 000 persons daily use the railroads of this Port District in making a round trip to and from New York City. For every one individual coming from Westchester County there are two from Long Island and four from New Jersey. Every one of these 500 000, if asked for his opinion of the ideal suburban transit arrangement, would probably be in favor of a rapid transit line passing within two blocks of his country home and proceeding without further stop to Manhattan Island and thence *via* a Broadway subway between the Times Square theatre district and Wall Street to a single stop near his office.

It is necessary to get a picture of the actual distribution of this commuter travel in Manhattan in order to provide means for taking care of it because it is not possible to continue indefinitely to handle suburban commuters in the city subway system.

Preliminary analyses of the passenger traffic at the Pennsylvania Station and Grand Central Terminal show that somewhat more than 80% of the commuters either walk or use the city subways to reach their destinations in Manhattan. (The walking habit appears to be more pronounced among the Long Island commuters than among the Westchester commuters). This shows two things: First, that if a suburban transit terminal facility is placed within reasonable walking distance, the commuter will walk to and from his place of business; and, second, that the intra-city subway lines are called upon to handle a heavy morning and evening concentration of short-haul traffic on the densest portions of their systems. In 1927, according to the Transit Commission, subway travel in New York City increased at the rate of 290 000 riders per day throughout the entire year—an appalling rate of growth.

Another important fact is the effect of location of transit and terminal facilities on geographical distribution of commuter traffic. It is probably a safe statement that the location of the commuter's home is more often determined by the location of his business than *vice versa*. Thus, a man in business on the west side of Manhattan is more apt to live in New Jersey or on Long Island which can be reached by the west side subway and the Long Island Railroad. Conversely, a commuter in business on the east side is more apt to live in Westchester County. The Suburban Transit Engineering Board has data to show that the commuter traffic of the Pennsylvania Station and Grand Central Terminal is, in total, about equally divided east and west of Broadway, but that the majority of the Pennsylvania Station commuters have their places of business west of Broadway, whereas only about one-third of those using the Grand Central Terminal go in that direction.

It is premature at this time to make any concrete suggestions for solving this problem since the studies now under way are proceeding on a scientific basis toward that end. The writer agrees with Mr. Stuart* that there must be a distribution system on Manhattan separate and distinct from the city

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1391.

subways. He is not prepared, however, to state at this time that such a distribution system should operate in and through Manhattan without transfer. Certainly, connections must be made with all the steam railroads, but whether these connections should be transfer stations or switch connections for the operation of through trains, are problems that require considerable further thought.

The cost of new facilities on Manhattan Island, as Mr. Stuart states,* will be tremendous. It is a serious economic question as to whether the two-hour morning and evening commuter traffic can and should bear this entire cost. It might be well to consider other possible sources of revenue in off hours such as mail, express, package freight, and long-distance passenger traffic.

The principal problem in connection with this whole question will be one of finance. In this there is room for all interested agencies to help. It is quite possible that the Port of New York Authority could be of material assistance in this financing program which, in turn, depends upon the amount of traffic and the additional fare to be charged. The latter is more or less limited by what the commuter will pay for the privilege of abandoning present facilities in favor of an improved service.

While the commuter, as the principal beneficiary, should bear the major part of the cost, the City and the surrounding counties can assist either by remission of urban taxes or by suburban benefit assessments. The City obtains improved transit service through the relief accorded its own lines, and the counties obtain a suburban transit system that will attract more and more population with the normal resulting benefits.

The complexity of the suburban transit problem of the Metropolitan District is apparent if from no other viewpoint than the numerous social, economical, and political units involved. The first thing that is needed is a practical physical plan. Such a plan when evolved and acceptable to all interests would then be the logical basis for discussion of a legal financial plan. This is the program on which the Suburban Transit Engineering Board is now working.

FRED LAVIS,† M. AM. SOC. C. E. (by letter).‡—This paper is probably the most complete exposition of the essentials of the New York commuter problem which it is possible to present in such a short space. The ramifications and details of the subject are almost infinite, but with the main outline as thus presented there can be little disagreement; nor, in the estimation of the writer, can there be disagreement as to the main outlines of the solution which the author proposes, in so far as this relates to the essential features of the construction in New York City and the connecting lines in New Jersey, Westchester, and Long Island.

The importance of the problem is not in the least exaggerated, nor is the indifference or ignorance of most of those concerned, whether it be that of Government officials charged with the welfare of the people, the transportation

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1393.

† Cons. Engr., Jersey City, N. J.

‡ Received by the Secretary, September 11, 1928.

agencies charged with the duties of moving them from place to place as they, the people, desire, or the people themselves. The author does well to emphasize the magnitude and importance of the problem and to strip this to its bare essentials in order to try to bring those who should be interested to a realization of what it really is.

As to his general scheme for construction, it follows more or less the plan proposed by Daniel L. Turner, M. Am. Soc. C. E., for the New York Transit Commission several years ago, and while, in general, Mr. Stuart very wisely avoids details of location, he does rather commit himself to the proposed 57th Street Bridge.* There seems to the writer to be several very serious objections to this, but it is a detail of the plan which does not affect its fundamental validity.

The question of finance is one of the utmost difficulty, far more so than the solution of the engineering problems which must be approached from the highest and broadest points of view; that is, not merely as to the development of details of construction, but from the point of view of moving, in the best manner possible, those who desire to move.

The difficulty of finance is probably best illustrated by the report of the Westchester County Transit Commission, rendered about 1926, which proposed the construction of an independent subway from 138th Street to Lower New York.

Briefly and generally, the Commission's thesis was that there were, in 1925, approximately 40 000 000 commuters arriving at or leaving the Grand Central Station yearly, from points in Westchester, and that by certain dates in the future, corresponding to the completion of certain sections of the subway proposed, this figure would be increased to 70 000 000 and later to 100 000 000 per annum.

It was assumed that with this number of passengers (70 000 000 to 100 000 000) a 10-cent increase in fare would produce additional revenue sufficient to pay the interest and amortization on the $150 000 000 which the subway would cost. The calculations, however, ignored completely the probability that the New York Central and the New York, New Haven and Hartford Railroads—on their existing lines and with their present facilities—could continue for some time to take care of as many commuters as they do to-day, and even though their through traffic should increase they could still probably take care of 30 000 000 to 35 000 000 per year for many years to come, thus materially reducing the number which it was estimated would use the new facilities.

The financial problem is also made difficult by the fact that any special commuter facilities in New York City would only be intensively used during the rush hours; that is, at the most, 3 or 4 hours per day. Experience has shown that facilities of this type must be used very intensively for at least 12 to 15 hours per day to provide an adequate return, even at a 7 or 8-cent fare. It must be remembered, also, in this connection, that outside the rush hours, there is already a tendency for people living in the suburbs to use buses, and this practice seems much more likely to increase than diminish.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1392.

The author also assumes that there will be an attitude of amiability or at least of fairness—on the part of the regulatory or rate-making authorities; but even where New York City is a partner in the enterprise it seems to be entirely indifferent to the economic aspects of this tremendous problem and seems to look at it entirely from a political point of view. It would seem to the writer that the author is unduly optimistic in his estimate of the attitude of the regulatory bodies and the ability to obtain the money for such an enterprise entirely from private sources. It is thought, therefore, that the State, the counties, the municipalities, and the railroads should all be partners in the enterprise, the Governmental authorities supplying a reasonable proportion of the basic funds, the remainder from private finance to have a prior lien on the profits.

These difficulties of finance are stressed, not because they are insuperable, but because they are a further illustration as the author emphasizes, that this problem is of very great magnitude, and can not be solved by any individual, entity, or section, but requires for its solution the utmost ability of a body of specially trained engineers, lawyers, financiers, and business men, of the highest caliber and broadest experience, acting with authority and backed by ample resources.

The difficulties developed by the plan of the Westchester Transit Commission are a very pertinent illustration of the practical impossibility of obtaining a solution of this problem by individual effort, even though this be as important as that of the County of Westchester.

Gustav Lindenthal,* M. Am. Soc. C. E.—The outline of a comprehensive system of suburban transit for the Metropolitan District has been given by the author, the essential feature of which is the entrance of the different railroad lines from the north, west, and east through deep tunnels large enough for standard railroad equipment.†

To avoid interference with the already existing or future rapid transit tubes, these suburban transit tunnels would go down to a low level and would have no direct connection with rapid transit tracks. Deep tunnels are feasible in the Metropolitan Area because of the underlying rock formation, except beneath the Hudson River, where the rock is from 200 to 350 ft. deep, overlaid with sedimentary formation forming the bed of the river 40 to 60 ft. deep. Such suburban transit, although independent of local rapid transit lines, should be in connection and co-operation with them. Access from these deep levels to the streets and to the subways would have to be by large elevators. Stairways would be only for emergency use. With large elevator capacity, trains and platforms can be emptied and loaded in the shortest practicable time. The cost of operating large and swift elevators is only a fraction of a cent per passenger. The exertion of climbing stairways would be spared the passengers—and this in itself would certainly be a boon to children and old people.

At the deep level, the network for this suburban transit can be arranged without great difficulty, so that all the railroads entering the Metropolitan

* Chf. Engr., North River Bridge Co., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1392.

District shall be connected one with the other for the interchange of trains. Additional tracks in the Metropolitan Area will be required on some railroad lines, but others, now mostly idle, will be more fully utilized.

The bridge proposed at 57th Street† will form an important part of this suburban railroad system, although it will have to clear the river at a height of about 175 ft. River tunnels, like those of the Pennsylvania Railroad for cars of standard size, would cost from 100 to 150% more per track than a multiple-track bridge, as designed for the 57th Street crossing.

Lanes for vehicle traffic are also cheaper over bridges than through river tunnels, in the proportion of 1 to 2. To the greater cost of construction of vehicular tunnels must be added the large cost of ventilation; but vehicular traffic is a problem separate from the suburban railroad transit system.

Since the bridge at 57th Street is planned to connect with all the railroads in New Jersey and, eventually, will have a large Union Station on the New Jersey side, it will form, really, the principal artery from the west. The bridge approach in Manhattan can readily be connected with standard-size tunnels extending south to a point near the Battery and thence under the river into New Jersey. Tunnels and bridge would thus furnish a loop connection in Manhattan for all railroads in New Jersey and also for those entering from the north and now terminating in the Grand Central Station.

The suburban system could thus give continuous train service day and night on all the railroads to a distance of 25 or 30 miles out from the center of Manhattan. There would be transfer stations at suitable points to the subway lines in every part of the Metropolitan District.

The principle on which Mr. Stuart insists,* that of keeping the suburban system separate from the rapid transit system, is important and necessary for obtaining the greatest economy in operation simultaneously with the greatest rapidity of train movement.

The cheapening of electrical power now possible through combination and equalization of peak consumption for light, transportation, and industrial purposes is an important factor in the solution of the problem.

Obviously, the successful consummation of such a system depends on a comprehensive and thoroughly prepared financial plan. The question is whether public credit should participate. After a careful study of all the factors of this plan, the speaker is of the opinion that a combination of the railroads would make the use of public credit unnecessary, but friendly and helpful co-operation of the public authorities is essential. The fare to be charged should not be a political football, but should be based on actual cost of construction and operation, including all other legitimate charges. A thoroughly developed financial plan is as necessary as a complete construction and operating plan. The author's paper—short as it is—gives a sufficiently clear idea of how such plans should be prepared for execution.

Charles Evan Fowler,† M. Am. Soc. C. E.—It is certainly a very notable event when an engineer of Mr. Stuart's knowledge and experience presents a paper on such an important subject, and has the courage to offer a solution

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1392.

† Cons. Engr., New York, N. Y.

of the complex problems involved in New York traffic. Any one who has spent several years, as the speaker has done, in studying these problems, begins to realize that it is almost a superhuman undertaking to solve the tangle.

As recently defined by the Port Authority, the Metropolitan Area has practically 10 000 000 people in it; and it will be only a few decades before there will be a population of 20 000 000 in a new and somewhat larger Metropolitan Area. If a great percentage of these people must be taken care of on Manhattan Island almost daily, it will become an absolutely impossible problem.

In traveling over the Metropolitan District, one becomes immediately impressed with the fact that the city is separating into districts where there are factories and business centers, so that the people can live near their work, and will come to Manhattan only when they desire to see the "White Lights" of Broadway. The speaker believes there will be an evolution in the next ten years which will change the whole problem, and the transportation facilities will not have to take care of nearly as many people on Manhattan Island as at present.

Several years ago the speaker made a suggestion in an article in the *Michigan Manufacturer*, that industries in Detroit, Mich., be located in the small towns around that city, to avoid creating the congestion found in other cities. Mr. Henry Ford, soon after that, adopted the same idea; and Detroit, perhaps more than any other city, is widely separating its industries, to avoid such traffic congestion as there now is in New York City.

Mr. Stuart's plan might solve the problem, but there is so much to be said for the plan of dividing the traffic in New Jersey and delivering it to Manhattan through the subways, as against bringing it all across to terminal stations and separating it there, that only an engineering commission, after extensive study, can determine exactly what is best.

The traffic volume and congestion is well illustrated by the money situation in New York. Some years ago the Federal Reserve Bank System was adopted. Every one said "that is going to solve the problem". Now, there are such vast sums handled daily in Wall Street and other business centers, that the Federal Reserve banks are at their wits' end to know what to do to control affairs, and are changing the discount rate almost daily.

A similar complex situation exists as regards the traffic in the district in and around New York City. The whole situation can be met and solved only by a co-ordination of all the different factors. It is evident to the speaker that much of the traffic in New York must be carried on the surface of the streets. To make this possible, and to do away with much of the congestions in the subways, the street cars and tracks should be eliminated entirely and numerous modern and large buses put into service. It seems to the speaker that only by such co-ordination, and change in the method and routing of traffic, can the surface traffic be taken care of in the future. Much of the through traffic can be taken around Manhattan, as will be the case when the 178th Street Bridge and the Tri-Borough Bridge are completed.

There will be needed very shortly one-way streets north and south on Manhattan. That is, Broadway and Fifth Avenue, perhaps, will be one-way south in the morning, and the opposite way north at night, with probably every other street two-way in order to make possible the right and left turns. The speaker does not believe that the railroads will take part in such a tremendous plan as Mr. Stuart has discussed. The only hope of doing it will most likely be by means of a private corporation, after comprehensive figures are available, which will demonstrate that the plan will pay.

The speaker has recently studied the surface transportation and has made a report on bus transportation lines, and also on some terminal propositions, and finds that the costs develop into such tremendous figures that it is almost impossible to predict what is going to happen by the time any plan is put into service.

One of the things that will help Manhattan Island is the West Side express highway which is now being started. Some such highway will soon have to be put in service on the East Side, and possibly something like the Henken plan, with an express roadway and rapid transit combined, for the whole length of Manhattan Island and Greater New York, might prove admirable if it can be financed; but for anything to be financed, data must be placed before the bankers to show that it will pay, and pay without being bothered unduly by legislation. Financiers are becoming wary of anything in the transportation line that can be hampered by politics.

It seems that a comprehensive plan must come about through some such organization as the Society; and until the Society takes the lead in getting a commission appointed, and seeing that engineers are put on the commission to study all phases of the problem, it is not likely that the problem will be solved. One can guarantee, by looking over the faces in the subway and on the elevated, that 90% of the people do not think a solitary thing about the fundamental principles of transportation. They simply pay their nickels to get their rides to where they want to go, and they do not know how it is done or why it is done. The masses do not think logically about it; but it is the business of the engineer to get them to see it rightly, and it is the business of the Society to take the lead in the tremendous problem of solving the traffic situation of Greater New York.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

EFFECT OF EARTH SHOCKS ON STRUCTURES

Discussion*

By Messrs. Rudolf Briske, Harry O. Wood, and R. R. Martel.

Dr. Ing. Rudolf Briske† (by letter).‡—The question as to what movements are experienced by a structure during an earthquake can be best studied by taking a mathematically simple case, namely, the vibration phenomena in a vertical free beam, considered as without weight and firmly fastened below, and with a horizontal shaking motion of the earth.

The maximum deflection, $Z_{max.}$, at the top of the bar may be found by the formula:§

$$Z_{max.} = \frac{e}{1 - \frac{T'^2}{T^2}} \qquad (5)$$

in which, the vibration period, T', of the bar under the load, P, is:

$$T' = \frac{2\pi}{\sqrt{3}} l \sqrt{\frac{l}{g}} \sqrt{\frac{P}{EI}} \qquad (6)$$

and e is the amplitude of the earthquake and T, the period of free oscillation.

Fig. 12 shows the influence of the change in the ratio of the vibration period, T', to the earthquake period, T. The results are made approximately applicable to structures that vibrate under their own weight, by substituting for the structure a rod of two-thirds the height of the structure, with a single load at the top corresponding to the weight of the rod.

For the practical evaluation of this condition, numerical definiteness on earthquake periods and natural vibration periods is necessary. The periods of oscillation, even with severe earthquakes, are at least 1 sec., and generally

* This discussion (of the paper by M. de Bussy, Esq., published in May, 1928, *Proceedings* but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Berlin, Germany.

‡ Received by the Secretary, August 31, 1928.

§ "Die Erdbebensicherheit von Bauwerken," von Rudolf Briske, Verlag Wilhelm Ernst & Sohn, Berlin, 1927, p. 25.

considerably longer. As is shown by measurements and calculations, the natural vibrations of structures at most are only small fractions of a second; only very high chimneys, very slender towers, and buildings with high slim columns without stiffening show natural periods of more than 1 sec.

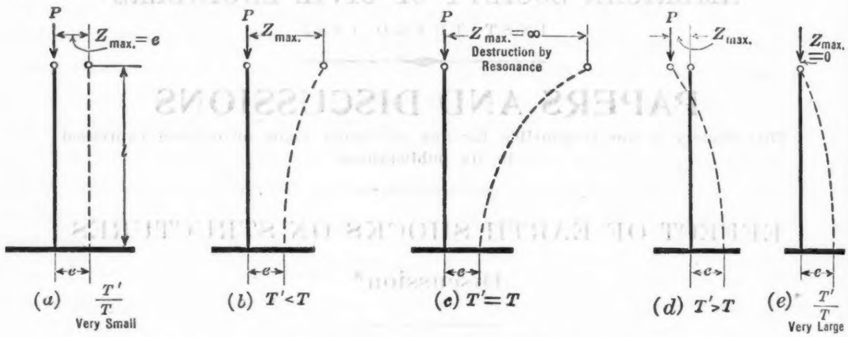

FIG. 12.—THE INFLUENCE OF A CHANGE IN THE RATIO, $\frac{T'}{T}$.

For almost all structures in earthquake regions it is practically sufficient to make this (see Fig. 12 (*a*)) the basis of the calculation; that is, consider a to-and-fro motion of the structure that in itself is rigid, and apply the usual method of calculation to combine the horizontal supplementary forces with the vertical forces in the same ratio as the horizontal earthquake acceleration bears to the vertical acceleration of gravity. The destruction phenomena in the great earthquake in Japan in 1923 and (so far as the writer knows) in all other destructive earthquakes, have shown that this method of calculation is practically sufficient and that only lack of strength against such supplementary forces in the usual construction methods was the cause of most of the destruction.

In the exceptional case mentioned, that of particularly slender constructions, which suffer deflections (see (Fig. 12 (*b*)), a recalculation of the stresses is necessary due to the deflections and any possible danger due to resonance (see Fig. 12 (*c*)). The condition shown in Fig. 12 (*d*) with a transition to the limiting case shown in Fig. 12 (*e*), is conceivable with high chimneys in earthquakes, but it does not enter the question of statical calculation. As an example, a reinforced concrete chimney 150 m. high with a 2½-sec. natural vibration period would not be endangered by an earthquake with a period of 1 sec., but would be destroyed by resonance by a less severe earthquake with a period of 2.5 sec.

The effect of the yielding of the ground which was discussed* at length by Mr. de Bussy may be dangerous with slender towers and chimneys, as a result of increasing the length of the natural vibration period; hence, there is increased danger from resonance. Tilting need not be feared in other structures, even with poorer foundations. In short, the effect of the bad ground is already considered in the statical calculation because the horizontal earthquake acceleration increases with the decrease in the quality of the

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1449.

ground. As an example, in Tokio, in 1923 the horizontal earthquake acceleration in alluvial river bottoms was 30%, and in diluvial raised areas only 10% of the gravitational acceleration.*

HARRY O. WOOD,† ESQ. (by letter).‡—Referring to the author's remarks on the "Motion of the Ground",§ it is true, of course, that structures are designed to withstand vertical accelerations greater than those in a horizontal direction and with greater factors of safety; and it may be true that the accelerations which occur in earthquakes are greater in the horizontal than in the vertical direction; but owing to difficulties in measurement and registration of motion in the vertical by means of instruments, it is not wholly certain that this is so. Emphasis should not be placed too strongly on the possibility that vertical earth motion is less energetic than the horizontal, and it should not be taken as a fact.

Visible waves of short wave lengths have often been reported. In some cases the reports are based without question on an optical or psychological illusion. In other cases the testimony would indicate that such visible short-length waves are real.

Again, with respect to the "Movement of Objects",‖ the writer is certain that movements of sliding, or the hitching along of objects, must unquestionably be facilitated when the motion of vibration has a vertical component, and this also would facilitate the causation of the apparent rotation. Further experimental work with reference to such action is desirable.

R. R. MARTEL,¶ M. AM. SOC. C. E. (by letter).**—If the characteristics of ground movements to be expected during an earthquake were known, the problem of determining the forces produced in a building or other structure would become definite and subject to at least an approximate solution. The dearth of information regarding the motion of the ground in the areas where earthquakes have caused destruction is a condition that seems bound to continue for some time to come. The reason for this is the comparative rarity of the occurrence of destructive earthquakes in densely populated regions and the somewhat general lack of interest on the part of seismologists in securing the data needed by engineers. It may be that general limits, such as the range of periods and amplitudes of the oscillations, are all that engineers may ever hope to obtain because of the absence of standardization in earthquakes even in a given locality. However, it should be possible to obtain data which, although it might not be extremely precise, would eliminate some of the current speculation as to earthquake motions. Until data of this kind are available, studies such as those of the author may provide explanations for certain observed phenomena which will give confidence in the assumptions of the character of the motion on which the studies are based.

* "Die Erdbebensicherheit von Bauwerken," von Rudolf Briske, Verlag Wilhelm Ernst & Sohn, Berlin, 1927, p. 25.

† Research Associate in Seismology, Carnegie Institution of Washington, Pasadena, Calif.

‡ Received by the Secretary, June 30, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 1449.

‖ *Loc. cit.*, p. 1451.

¶ Associate Prof., Civ. Eng., California Inst. of Technology, Pasadena, Calif.

** Received by the Secretary, September 11, 1928.

In his treatment of the effect of a single "to and fro" displacement with different acceleration patterns on a single mass mounted on a flexible rod,* the author presents an extremely broad field of research comparable with that of transients in electrical engineering. Some of the lines along which further studies in this field might be pursued advantageously are the effects of a number of oscillations of different acceleration patterns on the single mass supported on an elastic rod and on simple multi-story bents, as well as on an elastic rod of varying cross-section. The writer suggests that the author outline the simple approximations by which the results given in the paper have been obtained and by which some of the mathematical difficulties of the problem have been evaded.

The effects of damping have not been discussed by the author and rightly so, since they would probably not affect the qualitative results found, and their inclusion in these early stages of the study would lead to complications and perhaps to confusion. It might be noted, however, that the damping in a building is quite different from the damping in seismographs which is intentionally provided to prevent resonance. In a building the damping force is, in the main, a constant due to what is sometimes called "solid friction", and it decreases the amplitude of each successive free swing by a fixed amount finally leaving the structure permanently distorted after motion has ceased. The type of damping in seismographs is, in the main, proportional to the velocity of the moving mass.

The importance of rocking caused by the yielding of the foundations, to which the author devotes a part of his paper† has, also been discussed by Dr. K. Suyehiro, Director of the Earthquake Research Institute, Tokyo Imperial University, who made experiments with small models resting on gelatine which was given a simple harmonic motion by means of a shaking table. Like many other valuable papers on earthquakes and their effects on engineering structures, the report of these experiments is in Japanese.

The illustration (Fig. 5)‡ used by Mr. de Bussy to show how the yielding of the foundation of a chimney affects the location of the section of probable fracture, furnishes an explanation for the damage actually observed in many earthquakes. However, it is not the only explanation that can be found, since it may be shown that the location of the point of critical stress in a flexible rod subjected to simple harmonic motion may occur at any point along its height depending on the ratio of the free period of the rod to the period of the motion; but when this method is applied to the data on the chimneys in and around Tokyo broken by the earthquake of 1923, no correlation is found.

It may be mentioned here that the data on damaged chimneys which led Omori to suggest that the critical section of a chimney was at the center of percussion, because the average distance from the base to the fractured section of those which he had observed was two-thirds the height, included cases where the location of this section ranged from 0.25 to 0.95 of the height. The fallacy of using the average in this case is now obvious, but the fact that this

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1450.

† *Loc. cit.*, p. 1451.

‡ *Loc. cit.*, p. 1456.

average fortuitously fell at the critical section of a rigid rod subjected to a single transverse force at its base gave color to the assumption that in chimneys and in tall buildings subjected to an earthquake the stresses were like those in the rigid rod. The wide acceptance of this theory of the "center of percussion" which is current even to-day is another example of the indestructibility of error.

The author is to be commended for his stimulating contribution to a subject which has received too little sustained study in the past. The earthquakes of the past few years have provided a mass of data from full-sized structures available for the verification of the complete solution.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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ENGINEERING FOUNDATION COMMITTEE ON ARCH DAM INVESTIGATION

ARCH DAM INVESTIGATION

Discussion*

BY MESSRS. ROY M. GREEN, HARRY O. WOOD, AND R. A. SUTHERLAND.

ROY M. GREEN,† M. AM. SOC. C. E. (by letter).‡—This report contains a great fund of valuable information with respect to the problems of the use and durability of concrete as a structural material. The investigations still being continued by the Committee, should yield information of even greater value than that already published.

One significant statement appears in the report§ to the effect that under a stress of 216 lb. per sq. in., the test beam gave a secant modulus of elasticity of 4 000 000 lb. per sq. in. for the earlier tests as compared with 6 000 000 lb. per sq. in. for the later tests. Because the later modulus seemed too high, check tests were made and the value of 6 000 000 lb. per sq. in. was confirmed.

The very fact that check tests were thought to be necessary indicates a general impression that the value of the modulus of elasticity of concrete is a function of the strength and that age is not a controlling variable. This impression has doubtless grown out of the work of other investigators. According to Stanton Walker,‖ Assoc. M. Am. Soc. C. E., the relation between the modulus of elasticity and strength for mixtures leaner than about 1:3, may be represented by an equation of the form, $E = CS^m$, in which, $E =$

* This discussion (of the Report of the Committee of Engineering Foundation on Arch Dam Investigation published in May, 1928, *Proceedings*, Pt. 3, but not presented at any meeting), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Pres. and Mgr., Western Laboratories, Lincoln, Nebr.

‡ Received by the Secretary, June 1, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Report on Arch Dam Investigation, p. 154.

‖ "Modulus of Elasticity of Concrete," by Stanton Walker, Assoc. M. Am. Soc. C. E., *Bulletin No. 5*, Lewis Inst., Chicago, Ill.

modulus of elasticity of concrete; C, a constant depending on the conditions of the test; S, the compressive strength of concrete; and m, an exponent.

In studying these relations, especially as to the variation of the modulus with the strength and age as found by the Arch Dam Committee, the writer was interested in plotting a number of the results from Mr. Walker's previous tests. A typical result is given in Fig. 163.* Other curves confirming the same tendency might readily be shown. The test results are grouped into three curves according to their ages. While the general form of these curves is somewhat similar, the positions are noticeably different, which indicates clearly that, after all, age is as much a determining factor as compressive strength. For example, (Fig. 163) for a given type of concrete the modulus of elasticity corresponding with the compressive strength of about 5 000 lb. per sq. in. may vary from 5 000 000 to more than 7 000 000 lb. per sq. in., depending on whether the given strength is developed at 3 months or 1 year.

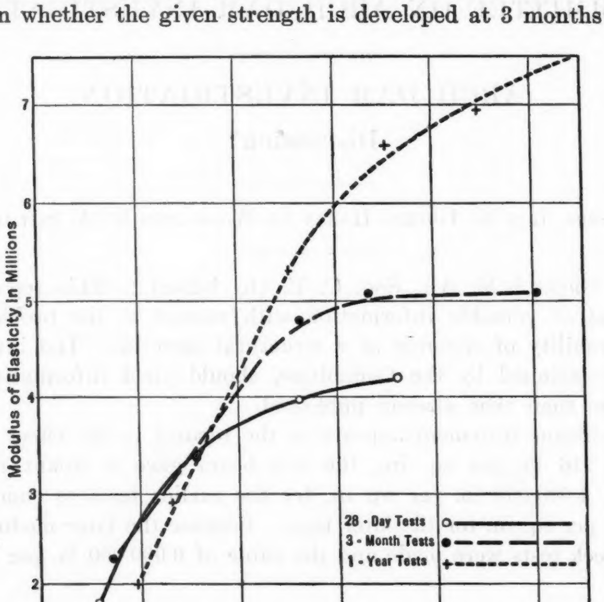

FIG. 163.—VARIATION OF E FOR CONCRETE ACCORDING TO AGE.

This seems to check exactly the tendency found by the Committee on Arch Dam Investigation. The fact that concrete has the characteristic of becoming brittle with age should be of sufficient interest to warrant further careful and complete investigation.

In the valuable investigation of the "Physical Properties of Concrete,"† by Raymond E. Davis, M. Am. Soc. C. E., it would be of value to record the unit deformation at the breaking load both for concretes that have been

* Plotted from Mr. Walker's Table No. 7.

† *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Report on Arch Dam Investigation, p. 199.

under a continuous loading and also for those that have been under no load during their curing and storage. This would determine whether or not there is the growth of brittleness in the concrete that has undergone plastic flow. Also, it would determine whether the plastic flow had materially reduced the total deformation at the time of rupture. While these are not such important factors in a dam, they are of great interest in connection with the use of concrete for paving purposes.

HARRY O. WOOD,* ESQ. (by letter).†—There is a possible action of earthquakes on dams, dikes, etc., which appears to have been generally, if not universally, overlooked. In an earthquake, the rock and solid earth particles which are held together by elastic forces—including the dam or other similar structure as part of the assemblage—vibrate to and fro in a complicated manner. However, the lengths of the earthquake waves are so great that this motion, in so far as the structure and the adjacent earth and rock material on which it is founded and built are concerned, may be thought of as a to-and-fro translatory motion of small amplitude. On the other hand, the water behind the dam, being a liquid of relatively small viscosity, may not and probably does not share fully in this motion until after a lapse of some time, if at all. There may be, therefore, a great inert mass of very small compressibility in contact with the practically vertical up-stream face of the dam. This mass may be more or less free from the horizontal or sloping earth or rock at the bottom and sides on account of the liquidity and low viscosity of the water and free, therefore, to a considerable extent to act as a great sluggish pendulum bob, or a kind of soft-nosed battering ram of relatively great mass, which tends to remain still as the earth and the dam shift rapidly to and fro. Relatively, then, a soft-nosed, but very heavy battering ram may be caused to deliver in rapid succession, blows, or thrusts, against the up-stream face of the dam.

Ordinarily, of course, the amplitudes of the earth vibrations are small, but in very strong earthquakes this is not true in the vicinity of the shock origin. Amplitudes up to several inches are confidently estimated to have occurred. While water does possess some viscosity, and in consequence is not wholly free to act as a pendulum mass, still, under such circumstances, it would seem that it may exert thrusts of great force. Consequently, this possible "water-hammer" effect ought not to be overlooked. Investigation may show that it is, or is not, negligible. There may be factors tending to counteract it.

R. A. SUTHERLAND,‡ ASSOC. M. AM. C. E. (by letter).§—This monumental work is deserving of the greatest praise from engineers and others interested in the construction of arch dams. It illustrates the enormous advance made in the knowledge of this class of structure in the last twenty years.

* Research Assoc. in Seismology, Carnegie Institution of Washington, Pasadena, Calif.

† Received by the Secretary, August 24, 1928.

‡ Chf. Engr. and Asst. Mgr., Christmas Island Phosphate Co., Christmas Island.

§ Received by the Secretary, September 6, 1928.

The V-shaped profile prepared for the test dam is a good average site for testing purposes, although many arch dams are built in sites of U-shape, and a few in sites of a cusped-shape, similar to a V, except that the side slopes are curved outward so as to form a very sharp gorge at the bottom.* This latter type is particularly adapted to dams of the constant-angle type. It will be found that the distribution of load between beams and arches will depend very largely on the shape of the dam site. This can be shown by trial and error analysis† and may now be said to be demonstrated in some measure in practice. The Stevenson Creek tests have demonstrated that for a V-shaped site, the arches, in general, are most heavily loaded in the center, while Professor Guidi, of Turin, has shown‡ by model tests that for a site of rectangular shape the arches, in general, are most heavily loaded at the flanks. It appears certain that for a cusp-shaped site, the arch loading would be even heavier in the central part than has been found to be the case at Stevenson Creek.

In the writer's opinion, one of the most valuable results of the work of the Committee will lie in the correlation of results obtained on the dam and on models, whether of concrete, celluloid, or other material. Such a correlation of full-scale and model tests would enable useful information to be obtained at small expense on the behavior of an arch dam in a site of any given shape. A testing laboratory equipped for model experiments of this type would then provide as great an improvement in arch dam design as the Froude tank has provided for British naval architecture.

The writer has shown elsewhere§ that the shape and dimensions of a dam site may be expressed roughly by a formula, and it is reasonable to expect that the load distribution may be expressed, qualitatively at any rate, in terms of the shape of dam, and that the best design may eventually be to some extent standardized for the different profiles.

One respect in which the Stevenson Creek tests do not correspond to general practice is in the sudden application of loads. With large dams particularly, the water load and temperature loads (although perhaps to a less extent) are generally applied slowly, and this must tend to mitigate to some extent the tensile stresses, such as those that caused cracking at Stevenson Creek.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1031, Fig. 1(*b*).

† *Loc. cit.*, p. 1027.

‡ "Studi Sperimentali su Costruzioni in Cemento Armato."

§ "Some Aspects of the Cost of Water Conservation," *Commonwealth Engineer*, June, 1928.

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PAPERS AND DISCUSSIONS

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STREET DESIGNING FOR VARIOUS USES

Discussion*

By Messrs. Harold M. Lewis, W. W. Crosby, W. C. Sawyer, William C. Reeder, and William T. Lyle.

Harold M. Lewis,† M. Am. Soc. C. E. (by letter).‡—This paper is encouraging in that it shows real progress in providing highway facilities to serve modern vehicular demands. It is valuable in determining the principles which should guide up-to-date highway design. The motor vehicle is no longer an experiment but promises to be a fixture for which the highway engineer must provide.

The writer would suggest another function for a street which deserves much greater consideration than is generally given to it; that is, a street should encourage and serve the most desirable kind of use that the owners of adjacent property can make of their land. The tendency has been to expect the frontages of all main highways radiating from a large city to be developed primarily for commercial purposes. Where zoning plans have been adopted they generally show such frontage in a business zone; or, where property is adjacent to railroads or navigable waterways, in an industrial zone. Much of such land develops very slowly if at all.

Before the intensive development of the motor vehicle, many miles of arterial streets on the edges of American cities were given over to high-class detached residences. Increasing traffic demands have caused the construction of additional arterial highways. Increased accessibility has brought an exaggerated idea of potential development and the real estate dealer has pictured practically all this frontage as future business sites. An application of the data given by Mr. Tillson will show the falseness of such a prophecy. The land owner and the real estate speculator have not tried to determine the

* This discussion (of the paper by George William Tillson, M. Am. Soc. C. E., presented at the meeting of the City Planning Division, Denver, Colo., July 14, 1927, and published in August, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Executive Engr., Regional Plan of New York and Its Environs, New York, N. Y.

‡ Received by the Secretary, September 2, 1927.

actual demands for business properties, but have held the avenue frontage at prices which have prohibited its use for detached residences. The result has been that improvement of such frontage has often been delayed until a sufficient number of residences have been built on the land fronting on the other streets in the vicinity to support local business establishments on the avenue. Such business then develops in strips of small stores. The community might be better served if it were concentrated in small business centers each of which cater to a separate residential area.

It may be claimed that a main arterial highway is not desirable for residences because of the noise and confusion resulting from the intensive use to which it is subjected. This is true if the street contains a trolley or elevated railway. It is not necessarily true of an attractive boulevard which may carry a heavy vehicular traffic but may still furnish excellent apartment sites. The question is, "Can a traffic way be made suitable for high-class detached residences?" The parkway (by which is meant a road through a park over which abutting owners do not have the right of light, air, and access) seems to offer a solution. Strictly speaking, such a traffic artery is not a highway, it is part of a park, but if laid out to take care of long-haul traffic it seems sufficiently a highway to be considered in this discussion.

If, instead of adding more roadways of the same kind between the older arterial routes, such parkways had been laid down through what was then relatively open areas, a much more efficient and economical arrangement might have resulted. Increased traffic capacity would have been added along attractive routes relatively free from interference with cross traffic and preserving as adjuncts to residential sites such natural charms as the country may have possessed. If properly protected by a zoning plan, which had provided sufficient sites for readily accessible local business centers, good residential development would have been attracted and maintained along the frontage of such parkways. That part of the Bronx River Parkway in Westchester County, New York, offers an excellent illustration of the successful execution of a project of this kind. In this case the development of adjoining property was greatly facilitated by the railroad which parallels the Parkway and furnishes necessary commuting transportation. A transit facility of some kind is essential to encourage development of such parkways. When they are within a few miles of the business center an adequate motor-bus service may prove sufficient.

The construction of parkways within areas already built-up may often entail prohibitive costs. There are still many places where they can be established economically on the outskirts of larger cities. The advantages that will accrue to both the highway system and the values of adjoining land are certain to make them good public investments. They will also provide "corridors" of open spaces which will help to maintain communities with local activities and interests and avoid the monotony in both appearance and living conditions which follows the unbroken spread of urban conditions into suburban areas.

Mr. Tillson has referred* to the super-highways which are being developed in different parts of the country on rights of way varying from 200 to 600 ft. in

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1733.

width. These wide rights of way have generally been proposed so as to permit a gradual expansion of roadway capacity or to maintain attractive surroundings. Other great advantages are flexibility in both capacity and arrangement of future roadways within them and the ease with which separation of grades at important intersecting highways can be attained without requiring additional property or damage to buildings in the vicinity.

The writer believes that more attention must be paid to the separation of highway grades so as to permit more uninterrupted traffic on main highways. The capacity of a highway can be approximately doubled by substituting uninterrupted movement for movement that is only intermittent because of the serious interference of cross traffic. In some cases grade separations may offer the cheapest method of taking care of additional vehicles; but even if the cost is more than highways of the old kind, the saving in time of travel and the gains in safety can be charged against the excess. The latter should bring a considerable reduction in the number of accidents. The great need on congested highways to-day is some means of making better average speed. The modern highway readily permits as great a maximum speed as seems desirable.

It is relatively easy to acquire wide rights of way in the open country between built-up communities. In order that these may function efficiently they should be linked with roadways of similar capacity which will by-pass the larger communities. The urgent problem is the acquisition of the necessary rights of way for such by-pass highways before the rapidly growing cities obliterate the gaps through which they might be laid down. Improvement of such routes can be done progressively as the traffic demands increase.

W. W. Crosby,* M. Am. Soc. C. E. (by letter).†—The writer ventures to suggest that the "design" of streets certainly comprehends consideration of their place or "location" as well as their "functions" and "use". In some cases the location may really be fundamental and the function and use adventitious. Sometimes, however, "function" or "use" may be allied with "location" so that the combination is fundamental to all else; and in almost all cases the "design" of any single street should be—as it often is not—considered as affected by the location of neighboring streets or as a design for a part of a system whether it be branch or main line.

It seems to the writer that nowadays lack of width is the most frequent criticism of existing streets. The author seems to realize the preponderance of this factor for he devotes more than one-half his paper to discussing widths and there are frequent references to the subject in other parts of the paper.

One of the latter is particularly interesting—the paragraph headed "Light and Air".‡ While the English Courts have always held that every one was entitled to "a place in the sun" in that country where naturally sunshine has been at a premium, and hence have prevented—by refusing to allow a neighbor to be shut off from his proper amount of sunshine (and air) through the erection of "skyscrapers"—the development of street canyons in English cities comparable to those of New York, for instance, American practices and controls have brought about results that are beginning to be recognized as involv-

* Cons. Engr., Coronado, Calif.

† Received by the Secretary, September 2, 1927.

‡ *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1726.

ing seriously objectionable features. The automobile traffic, with its by-products, has accentuated these objections.

Mr. Tillson writes* of the necessity for "bolstering up" by zoning the imperfect planning of street systems, and in a way there seems an analogy in the "setting-back" of upper stories of high buildings where insufficient street widths have been established. However, there is a real rational motive for both zoning and setting-back although often the effects are but palliatives instead of cures for the trouble.

In this connection it may be pertinent to invite serious attention to a suggestion recently advanced, which, if properly developed, may materially affect "street design". Excessive height of buildings is objectionable from many of the viewpoints of street authorities. Irrationally excessive heights might be checked and even controlled, by establishing zones of taxation on improvements (structures) rising higher than a primary zone at street level. "The higher the zone, the higher the tax" might not be an unjust slogan. The "rights" of the land owner would not be denied, but the "sun-and-air hog" would be obliged then to compensate, to some extent, the neighbors and the street users, and, in a way, the street designers.

Space between fronts of buildings across narrow streets may be obtained by the establishment (and enforcement) of laws for setting-back buildings, but there may be many cases where one or both of these powers may prove lacking. It might be better to provide the proper width for the public right of way in the first place and then trust to "Nature to take her course" for the results advised by the author.

Since the World War people generally (especially Americans) have developed an objection to "queues". Motorists, as well as other humans, are gregarious, but even "road snails" will avoid joining in long files, by spurting to pass, or by taking any parallel route available. Three-lane roadways, the writer believes, are objectionable. Two or four-way roadways (in addition to any parking lanes allowed) are needed for safety and modern traffic. Where there are no street-car tracks, the minimum width for a two-lane roadway with parking seems to be 32 ft. between curbs. Hence, the writer thinks that Mr. Tillson's suggested roadway width† of 24 ft. "in comparatively narrow streets" is not sufficient for general adoption.

The writer has previously expressed himself in agreement with Mr. Tillson as to the delivery of goods "within the warehouses" themselves. The time will come when the accepted solution of many street problems, arising from restricted widths and affecting street design, will include: Railway location below the ground surface; vehicular roadways on the surface, with parking and loading spaces in congested sections provided by the private interests involved; footways on platforms above the roadways; and airways above all.

W. C. Sawyer,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The points which the writer wishes to emphasize are:

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* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1729.

† *Loc. cit.*, p. 1731.

‡ Asst. Engr., City Engr.'s Office, Los Angeles, Calif.

§ Received by the Secretary, September 2, 1927.

1.—The great importance of early adoption or correction of street grades;

2.—The adoption of general standards for the maximum grades desirable for automobile or general traffic;

3.—The proportion of the total street width to be devoted to sidewalk and park use; and

4.—The necessity for more adequate and valuable data on the location of underground structures.

The City of Boston, Mass., is always referred to as a "well-known example" of poor street location. The alibi is its very ancient beginning. Is it not a disgrace for more modern cities, rapidly developing in the light of Twentieth Century intelligence to cling to methods that were a disgrace when the City of Boston was started?

Los Angeles, Calif., is in some respects a backwoods town. Until within very recent years, it has been almost wholly developed by sub-division projects. New tracts adjoining the original city limits were laid out with slight consideration for the continuation of street lines.

Of late years much attention has been paid to the matter by city and county governments. Millions of dollars are now being spent to ameliorate the trouble, whereas ideal conditions could have been obtained at the beginning with slight expense.

This is not, in the main, intended as a criticism of the engineer, although he is not blameless. It is the general system that is wrong. The city planing commission assumes no responsibility for street grades and the widths of roadways and sidewalks. The commission limits its responsibility to zoning and street alignment. Even in that it is doing a great work. It is to be admitted that it needs the integrity of a deity, the strength of a Samson, and the wisdom of a Solomon to solve successfully these problems alone. There is probably no street in the city which has not some advocates for changing it to a major traffic highway, a business or manufacturing way, or a boulevard. Some expert body should be available to study the possible early adjustment of grades and decisions as to permanent street widths before expensive construction is undertaken.

The writer suggests four specifications for street grades, as follows: (1) Grades less than 10% shall be classed as desirable; (2) grades from 10 to 15% shall be considered questionable; (3) considerable expense shall be justified to prevent or correct grades between 15 and 20% (because of safety and future property values); and (4) grades between 20 and 35% shall be considered prohibitive and justified only by extreme emergencies and under extremely favorable approach conditions.

There seems to be a fallacious opinion that almost any cut is allowable in establishing a grade, but that all fills are prohibitive notwithstanding the fact that fills are being made on private side-hill lots where the natural slope is much more than 20 per cent. The advisability of flattening the approaches to property that has a hilltop view by filling at the bottom is being overlooked. Where the streets in question are in business and residential districts where

basements will be desired, the elevation of the street will be a positive benefit by saving of excavation, hauling, and dumpage, and hillside lots sloping toward a frontage on such filled streets will profit in the same manner.

With regard to underground structures no carelessness of location is excusable on the ground that they are "out of sight, out of mind". The opposite attitude should be adopted; special care should be exerted to maintain convenient and accurate location data. Every underground structure should be located by its elevation above or below city datum and not by its depth, because changes in surface levels invalidate such records.

William C. Reeder,* M. Am. Soc. C. E.—The engineers for the Delaware River Bridge designed six lines of traffic $9\frac{1}{2}$ ft. wide. They were not encumbered with the need of providing for standing lines of vehicles. Consequently, they made the lines of equal width. It was found almost immediately that the buses and trucks, which had been delegated to the outer lines of traffic, would not confine themselves to those lines. Therefore, the outer lines were changed to $10\frac{1}{2}$ ft., and the inner ones reduced to 9 ft., which has proved quite satisfactory. A volume of traffic, amounting to 59 000 vehicles on some days, moves freely at speeds from 22 to 35 miles per hour, and the management believes that 100 000 cars per day can be handled satisfactorily.

Another phase of the question of street widths and lines of traffic, is that the dimensions of motor vehicles are continually changing. Streets designed for present-day conditions are found to be inadequate for to-morrow; for instance, the Pennsylvania Public Service Commission requires 14 ft. of headroom for a highway bridge, whereas the Motor Law, adopted in 1927, allows motor vehicles 14 ft. 6 in. in height. These vehicles are now allowed to be 8 ft. wide. What becomes of the 8-ft. line of travel formerly used by some cities?

Partly to overcome this question of changes, the speaker wishes to call attention to what is known as the "elastic" street ordinance, which the City of Philadelphia adopted in 1916 for the economic development of streets. A street may be opened legally for its full width between building lines, with the establishment of a curb line; but in the improvement of this street any reduced cartway width which economizes in the paving of the cartway can be used. The Board of Surveyors approves a plan for that individual street, temporarily allowing such development but, at the same time, fixing the line of the improvements that shall be made. The trees must be in a certain line, and the portion of the footway which is paved must conform to a uniform width and line, so as to be harmonious and fit the ultimate development of that street. Whenever increase of traffic may require it, the city can reset the curb back to the true line and pave the full cartway, assessing abutting owners for the portion not previously paved.

An Allocation Committee, consisting of a delegate from each of the public service corporations and the Highway Supervisors, has decided to recommend for all new streets that the secondary gas and water pipes for local service shall be placed in the footway, the gas pipes 6 ft. from the house line and the

* Asst. Chf. Engr., Bureau of Surveys, Philadelphia, Pa.

water pipes, 8 ft. This is done because of the frequent cutting of the cartway pavement, due to the necessity of giving private service, and the fact that it is somewhat easier to restore footway pavement than the cartway.

WILLIAM T. LYLE,* M. AM. SOC. C. E.—In the matter of street design the engineer must steer his course between Scylla and Charybdis—elasticity on the one side and stability on the other. With changing conditions, such as are now being experienced, a treatment capable of modification is evidently desirable. The rigidity of the plan of Manhattan Island is most unfortunate, although the planners of a century ago can hardly be blamed since the traffic in those days was from river to river. On the other hand several of the modifications of the plan of Washington, D. C., have proved themselves unfortunate and have subsequently been corrected by restorations. Great wisdom and foresight are necessary in preparing original plans in order to provide for the possible incorporation of needful modifications, while at the same time creating an initial rigidity making it difficult for future generations to destroy the harmony inherent in the original plan.

Mr. Tillson has called attention† to the attractiveness of planning for capital cities, where the probability of future changes is reduced to a minimum. He also calls attention to the artifice of zoning to create stability.‡

The combination of parks and streets or boulevards is to be recommended in providing for elasticity. This combination can take form in the modern parkway which is an elongated park with automobile and service roadways, together with planting strips and occasionally a bridle-path. Should future conditions require it, the planting spaces can be transformed into pavement, as was done on Main Street in Houston, Tex.

Attention has been called to a proper control of traffic lanes whereby their use is altered in direction of traffic to provide for the morning and evening flow.§ This, too, is in the interest of the principle of elasticity.

There is great need of giving more attention to through traffic—automobile, motor-bus, and motor truck. As far as the motor tourist is concerned, great differences are to be found in cities along the National trails. The able routing and traffic control in the City of Indianapolis, Ind., is admirable. The motorist is passed through without danger or inconvenience and in a remarkably short time. This is made possible by directness, by boulevard stops, by lights, and by police control in the congested district.

Sufficient attention has not been given in America to the aesthetic advantages of streets which are slightly curved. Streets of this character may even prove desirable in the closely built-up districts in order to afford a view of monumental buildings.

Mr. Tillson points out the need of taking care of future public services.§ Here the fundamental principle of harmony and common sense should come into play. Although public services are created for profit, the city cannot

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* Prof., Civ. Eng., Washington and Lee Univ., Lexington, Va.

† *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1727.

‡ *Loc. cit.*, p. 1729.

§ *Loc. cit.*, p. 1730.

get along without them, and every legitimate and proper assistance should be extended to them in the city plan.

The speaker is not convinced of the desirability of one-way streets for through traffic. This may be necessary sometimes. On the other hand, the routing of all through traffic on one street usually is a help to business. The by-passing of such traffic is most expedient in the case of a great city like Philadelphia, Pa., facilitating traffic and keeping the city's streets free for local business.

The foresight of the municipal authorities of Chicago, Ill., in securing rights of way for its boulevards before the price of land becomes prohibitive is to be commended. The wisdom of this procedure is capable of demonstration in several American cities.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

FINAL REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON FLORIDA HURRICANE

Discussion*

By Messrs. F. O. Dufour and T. L. Condron.

F. O. Dufour,† M. Am. Soc. C. E.—The report of the Committee is good. It covers all the points in an orderly and concrete manner. The speaker made an inspection of the effects of wind on buildings at Miami shortly after the storm.

The storm was as violent as any that has been registered; and on account of its passing over a city with many modern structures the opportunity for constructive criticism of the effects on usual engineering construction in areas of high wind velocity is exceptional.

What the actual velocities and pressures were appears to be immaterial, but nevertheless interesting. The speaker checked Clyde T. Morris, M. Am. Soc. C. E., as to the pressure when computed from the bending of columns in the front bent of the Meyer-Kiser Building,‡ basing the wind distribution on the usual assumptions. This pressure was 60 lb. per sq. ft. Whether or not this was the pressure or whether the effects were due to synchronous behavior or those of impact due to sudden failure of the curtain walls is beside the question. The fact that the greatest destruction of the columns of the Meyer-Kiser Building came after the walls were out appears to indicate that the bending was not due to the sudden failure of the walls. If the columns of the Meyer-Kiser Building had been designed for pressure of 60 lb., even with the details used, it is most probable that no material distortion would have occurred.

The fact that wind-bracing in the form of knee-braces was used below the fourth floor and that no column distortion occurred below that floor, appears to indicate that had wind-bracing of this type been used throughout no column distortion would have occurred.

* This discussion (of the Final Report of the Committee of the Structural Division on Florida Hurricane, presented at the meeting of the Structural Division, New York, N. Y., January 19, 1928, and published in August, 1928, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., United Engrs. & Constructors, Philadelphia, Pa.

‡ *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1758.

That the floors and ceilings were not even superficially cracked and that the rear of the building "kicked out" in a direction opposite to the front indicates that, as horizontal girders, these floors distributed the wind to the vertical bents in accordance with the usual supposition. Had the vertical wind bents, however, been equally spaced and had the wind pressure been uniform, it is probable that less damage would have occurred. That the wind pressure should have been uniform was, of course, not to be expected; that the strength of wind bents should be proportioned to stand a maximum difference in pressure is evident.

The report most properly calls attention to the insufficiency of details. The speaker noted a few broken connections and clip angles, but not more than three. A few more were cracked; many were bent.

In the case of the broken clip angles, one showed neither reduction of area nor elongation. Evidently, this break was due to causes other than the storm. Most of the bent connection angles showed no scaling. This indicated that the material was not stretched beyond the elastic limit. The partial springing back of the building when the walls were removed also indicated this.*

The stiffness of structures in areas liable to high wind velocity is, according to the report, next to strength, the most important consideration. The Committee has covered this in an efficient manner. Adequate interior wind bents and some construction for interior partitions other than plain partition tiles, together with a type of gusset-plate or knee-brace construction for the spandrel bents, are certainly required, especially when the depths of the spandrel beams are small.

An extended experience in the design of structures requiring stiffness as well as strength prompts the speaker to suggest that a design based on a unit stress of 9 000 lb. per sq. in. in tension, properly decreased for compression and increased for combined loads, will (in cases when the ratio of width to height is one-tenth or less) provide the desired stiffness, due regard being given to details sufficient to develop full strength.

It is believed that buildings designed in accordance with any of the methods commonly used by the leading structural engineers—a wind pressure of 30 lb. per sq. ft. and a due regard to details and wind-bracing—will, even with the present prescribed unit stresses, withstand injurious distortion as far as the steel frame is concerned. Consideration must be given to the element of stiffness, however, to prevent injury to the walls or inconvenience to the occupants. The floors will take care of themselves.

T. L. Condron,† M. Am. Soc. C. E.—It is stated‡ that if a wind load of 20 lb. produced a stress of 24 000 lb., a 30-lb. wind load would produce a 36 000-lb. stress. The speaker believes that the 24 000-lb. stress was probably due to combined forces and that the difference between the unit stress of 24 000 lb. and 36 000 lb. would be due entirely to the additional wind load.

* *Engineering News-Record,* E. A. Stuhrman, M. Am. Soc. C. E., December 29, 1927.

† Cons. Structural Engr. (Condron & Post), Chicago, Ill.

‡ *Proceedings,* Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1762.

MEMOIRS OF DECEASED MEMBERS

JOHN TONER BARR, M. Am. Soc. C. E.*

Died June 26, 1925.

John Toner Barr was born in Philadelphia, Pa., on November 25, 1876. He was the son of John C. Barr, of Blairsville, Pa., whose great-grandfather crossed the Allegheny Mountains from Baltimore, Md., long before the Revolution. His mother was Sarah Toner, of Gallipolis, Ohio. Her people came from York County, Pennsylvania, where they had settled during Colonial days.

Mr. Barr attended the public schools of Pittsburgh, Pa., until he was sixteen years of age, when he went to work as Draftsman in the City Engineer's Office. At the end of seven years, in 1898, he gave up this position and became a student in the Western University, now the University of Pittsburgh. He continued his studies at the University, working part of the time as Draftsman for the West Virginia Short Line Railroad Company, now part of the Baltimore and Ohio System, and also for the Allegheny Valley Railroad Company. He was unable, however, to finish his course at the University.

From 1899 to 1902 Mr. Barr worked, successively, as an Engineer for the Allegheny Valley Railroad Company, the Baltimore and Ohio Railroad Company, the Illinois Steel Company, and for the George S. Davison and the Mellon Interests. He designed the plant for the Pittsburgh Reduction Company at St. Louis, Mo., and was also Chief Engineer for the construction of the many miles of interurban electric railways running out of Pittsburgh.

Hard work and close application to the task in hand while fitting himself for his later undertakings began to undermine his health and he was advised to go West for recuperation. While in the West he worked in the office of the City Engineer of Salt Lake City, Utah; for a brief period as Engineer in Thunder Mountain, Idaho; later, at Pitkin, Colo., on mine surveys; and also as Machine Helper on the Oregon Short Line Railroad.

Mr. Barr returned to Pittsburgh in 1904 and became Chief Engineer for his brother, J. Carroll Barr, M. Am. Soc. C. E., and also for the Thomas McNally Company. He remained with them until 1907 when he launched out for himself under the firm name of J. Toner Barr, Civil Engineer, and from 1911 until the time of his death, practiced as J. Toner Barr, Engineer and Contractor. During this time Mr. Barr laid practically all the underground cables for the Bell Telephone Company in the Pittsburgh District.

On the evening of January 6, 1922, while at work in his office, a pistol which he had carried with him for many years for payroll protection accidentally slipped off his desk and was discharged. The bullet struck his right leg, glancing upward, necessitating, five days later, amputation in order to save his life. Mr. Barr met the situation bravely, declaring that he never had made any money with his legs anyway and proceeded to do more business following his accident than he had ever done before. He carried on until June 26, 1925, when he died after an operation for gallstones.

* Memoir prepared by Stewart C. McFarland, Esq., Pittsburgh, Pa.

In 1905, he was married to Anne M. Hogan, of Pittsburgh, who, with four children, Sarah, Rosemary, Jone, and Nellie, survives him. His brother, J. Carroll Barr, also survives him.

Mr. Barr was a popular member of the Rotary Club of Pittsburgh, holding the office of First Vice-President at the time of his death. The following memorial, delivered before the Club by Past-President and Past District Governor Stewart C. McFarland, who knew him intimately, gives a true picture of Mr. Barr and the esteem in which he was held by his fellows:

"Our friend, our First Vice-President—your friend and my friend—J. Toner Barr, has taken his place in the silent halls of cherished memories.

"Toner's going brings us once again face to face with the Great Inevitable—the final and conclusive fact that no matter how brilliantly a race the white steed of life may run, the black and inexorable steed of death is sure to win.

"When we laid Toner's tired frame in his narrow cell in Calvary last Monday morning we gave back to earth her own. His unconquerable soul and indomitable spirit, his clean life and splendid character, his sense of justice and his loyal devotion to truth, honor, and right are ours forever. This he has bequeathed to us and a richer heritage no man can bequeath to his fellows.

"Toner had a wonderfully discriminating mind and was frankness personified. He never ran away from any situation. He could detect insincerity afar off. He had a keen sense of values. He was seldom if ever misled by the false. He had the hard head of the engineer and the soft heart of the poet. At one time he remarked, 'Take sentiment out of life and all you have left is arithmetic.'

"Toner's home life was a symphony of sentiment. No man ever loved more tenderly his wife and children and no man ever lived in a richer atmosphere of love returned. The happiness of his home life was so complete that at one time he made the statement that he could be happy in Hades.

"J. Toner Barr's seat here to the right of President Jax is now vacant, but in the words of Riley:

" 'We cannot say, and we will not say,
That he is dead—he is just away;
With a cheery smile and a wave of the hand,
He has wandered into an unknown land.'

"To consecrate his memory and to render him a final Rotary service, I propose that we stand for a moment and in that moment pray for the well-being of his soul, and that in our prayer we include his bereaved and broken-hearted widow and children."

At the time of his demise Mr. Barr was an active member of the Chamber of Commerce, the Americus Club (since disbanded), the Engineers Society of Western Pennsylvania, the Historical Society of Western Pennsylvania, and the Duquesne Club of Pittsburgh.

Mr. Barr was elected a Member of the American Society of Civil Engineers on March 4, 1913. He was also a Past-President of the Pittsburgh Section.

WILLIAM TITCOMB BLUNT, M. Am. Soc. C. E.*

DIED MARCH 3, 1928.

William Titcomb Blunt, the only son of Aaron Drew and Helen Maria (Titcomb) Blunt, was born at Norridgewock, Me., on January 10, 1855. He

* Memoir prepared by the following Committee of the Illinois Section: John W. Woermann, Clifford C. Muhs, and Edward J. Fucik, Members, Am. Soc. C. E.

received his early education in the Grammar School and Eaton Family School of his native town, entering the Massachusetts Institute of Technology in 1870, from which he was graduated in 1874 with the degree of Bachelor of Science in Civil Engineering.

During the summer vacations of 1872 and 1873, Mr. Blunt was employed by the City Engineer of Somerville, Mass., and by others, on topographic and hydrographic surveys. Following his graduation in 1874, he secured a position with the U. S. Coast and Geodetic Survey as Observer with a party making a survey of the mouths of the Mississippi River.

From 1875 to 1879, Mr. Blunt served as Clerk and Assistant Engineer with the Harbor Commission of Massachusetts on triangulation, topography, hydrography, and mapping. From 1879 to 1883, he was engaged, for the Mississippi River Commission, on triangulation, topography, and hydrography along the lower sections of the Mississippi River. In 1883 he became associated with Mr. John Eisenmann under the firm name of Eisenmann and Blunt, at Cleveland, Ohio, in the practice of architecture and engineering.

From 1886 to 1890, Mr. Blunt was employed under the U. S. Engineer Office at Cleveland, as Inspector on the maintenance and improvement of the harbors along the south shore of Lake Erie. During the last six months of 1890 he was Principal Assistant Engineer in Charge of Surveys for the Sanitary District of Chicago, Ill., but resigned with the Chief Engineer, the late Lyman E. Cooley, M. Am. Soc. C. E., on account of their objections to the way the work was being conducted by the Board of Trustees.

For a time Mr. Blunt was busy in the insurance business, but the work was not congenial to him, and he soon returned to engineering. From 1892 to 1906, he was continuously employed as U. S. Assistant Engineer in the maintenance and improvement of harbors and the location and construction of lighthouses along the south shore of Lake Erie. This work was under the U. S. Engineer Office at Cleveland, but in immediate charge of the Sub-Office at Toledo, Ohio, where Mr. Blunt resided during this period.

From 1906 to 1907, he served as Superintendent of construction with George H. Breymann and Brothers, Contractors, of Boston, Mass., on river and harbor work, especially on subaqueous rock excavation, and, from 1908 to 1910 he was engaged on similar work for the U. S. Government at the Pacific entrance of the Panama Canal. In connection with this work he designed a rock crusher that was said to be superior to the Lobnitz crusher, which was also in use.

During the following two years Mr. Blunt remained on his farm in Ohio, but in November, 1912, he became General Manager for the Shirley-Skeldon Dredging and Submarine Contracting Company, of Toledo, with which he remained until May, 1914.

From June, 1914, until his retirement from the Federal Civil Service, in January, 1925, on account of having reached the age limit prescribed by law, he was employed as Assistant Engineer, under the U. S. Engineer Office at Chicago on the maintenance and improvement of the harbors at the south end of Lake Michigan. This work included dredging operations in eight rivers and harbors, rubble mound and concrete caisson breakwaters at Chicago Harbor and Indiana Harbor, and the construction of a concrete superstructure on the

old timber breakwater at Calumet Harbor. For a short period during the World War he was Acting District Engineer in charge of the office.

After his retirement from Government Service, Mr. Blunt was engaged by the Universal Portland Cement Company as Consulting Engineer on the construction of a new harbor at Buffington, Ind., including 1 300 ft. of rubble-mound breakwater. Following this, he was retained by the D. P. Davis Company, Realtors, as Staff Engineer in charge of extensive dredging operations at St. Augustine, Fla., on the development of a new seaside sub-division.

After the collapse of the Florida land boom in 1927 he returned to Chicago, and became affiliated with the Division of Waterways of the State of Illinois. During the summer and fall he was in charge of the repair of levees along the Mississippi River between St. Louis, Mo., and Cairo, Ill., and during the winter he was engaged in miscellaneous office work in Chicago.

Mr. Blunt was extremely thorough and careful in his work, and remained active and energetic until eight days before his death, which was due to pneumonia following a cold. He made loyal friends wherever he was located, and enjoyed nothing more than an evening at cards or story-telling. He has been characterized by one of his friends as "a rare raconteur tempered by the fire of a crusader; a *bon vivant* tempered by a fine æstheticism; a true friend, and a fair opponent."

Mr. Blunt was twice married, first, in 1874, to Frances Sydney Gibbs, of Toledo, Ohio, who died in 1907, and then, in 1909, to Helen Elizabeth Norton, who survives him.

Mr. Blunt was a Royal Arch Mason and a Charter Member of the Royal Arcanum of Cleveland. He was a member of the American Association of Engineers and also of the Western Society of Engineers. At the monthly luncheons of the M. I. T. Club of Chicago, he was usually the oldest graduate present.

Mr. Blunt was elected a Member of the American Society of Civil Engineers on October 5, 1898.

WALTER WOODBRIDGE CLIFFORD, M. Am. Soc. C. E.*

DIED JANUARY 19, 1927.

Walter Woodbridge Clifford was born on February 15, 1888, at Wiscasset, Me. His parents died when he was a small boy, and he went to Hyde Park, Mass., to live with his uncle and aunt, who raised him. He attended the public schools of that city and was graduated from the Hyde Park High School in 1905. He then entered the Massachusetts Institute of Technology at Cambridge, Mass., receiving his degree of Bachelor of Science in the Department of Civil Engineering in 1909. While in attendance at the Institute he was a member of the Civil Engineering Society, and took an active interest in student military affairs.

* Memoir prepared by Charles R. Berry, Assoc. M. Am. Soc. C. E.

In the summer of 1909, following his graduation, Mr. Clifford was in the service of the old Harbor and Land Commission of Massachusetts, as Surveyor in connection with the proposed dredging of harbors along the Massachusetts Coast. In the fall of that year he returned to the Massachusetts Institute of Technology as an Assistant in the Department of Civil Engineering, where he remained for two years. During the summer of 1910 he was employed as Resident Engineer by the Aberthaw Construction Company, which was building a concrete stand-pipe for the Town of Westerly, R. I.

After two years of teaching, he entered the employ of the United States Forestry Service as a Hydraulic Engineer, but he remained in the Service only a few months. From May to September, 1912, he was employed by Metcalf and Eddy as Resident Engineer in charge of the water supply and sewage disposal of the new Technology summer surveying camp, which was then being built at East Machias, Me. On the completion of this work he was employed for a few months by the U. S. Navy Department, at the Charlestown Navy Yard, as a Designing Engineer.

In 1913, Mr. Clifford entered the Engineering Department of Stone and Webster, Incorporated, and was assigned to the staff selected for the engineering design of the new Technology buildings at Cambridge. He did some valuable work in the solution of many difficult problems which arose in connection with the foundations and superstructure of these buildings. In 1915 he was employed as a Structural Engineer by Densmore and LeClear, and remained with this architectural firm for about two years. From 1917 to 1921, he was with the Harry M. Hope Engineering Company, first, as Chief Draftsman, and, later, as Designing Engineer on many important structures, among which was the large plant of the United Drug Company, at St. Louis, Mo.

From 1921 until his death, which occurred suddenly from heart failure at his office in Boston, Mass., he had been associated with Mr. S. G. Roeblad, under the firm name of Clifford and Roeblad, Consulting Engineers. The principal work of this organization consisted in designing industrial plants and garages, and acting in the consulting capacity of structural engineers for architectural firms. Mr. Clifford was engaged personally as an expert in Court proceedings, and also as arbitrator in connection with the interpretation of building contracts.

He was a member of the Boston Society of Civil Engineers and the American Concrete Institute. He was the recipient from the Society of the Collingwood Prize for Juniors, for his paper, "A Reinforced Concrete Standpipe."* He took an active part in the Boston Society of Civil Engineers, having served on many of its committees. He was Chairman of the Designers Section in 1923 and 1924. At the time of his death he was Chairman of the Welfare Committee and was a candidate for Director of the Society.

Mr. Clifford was the author of many articles on engineering subjects appearing in periodicals, and was one of the Associate Editors of Hool and Johnson's Handbook of Building Construction, and of Hool and Kinne's Structural Engineers' Handbook Library. With C. Hale Sutherland, M. Am.

* *Transactions*, Am. Soc. C. E., Vol. LXXIV (December, 1911), p. 375.

Soc. C. E., he had recently published a textbook entitled, "Reinforced Concrete Design," which was very favorably received.

Not only was Mr. Clifford active in the affairs of his profession, but he also gave generously of his time to Hyde Park Lodge, A. F. and A. M., and Christ (Protestant Episcopal) Church, of Hyde Park, of which he was a member. At the time of his death he held the office of Junior Warden in each of these bodies. He was also a member of the Aleppo Temple Shrine of Boston. He was a Director in the Hyde Park Young Men's Christian Association.

In 1912, he was married to Helen Edelmer Mooar, by whom he is survived. He also leaves two children, James and Deborah, as well as an only brother living in California.

Although of an unassuming disposition, Mr. Clifford took an active part in the various organizations with which he was connected, and was sought out by them for office on account of his fine executive ability. A man of accomplishments, and a true Christian gentleman, Walter Clifford will be missed by a host of friends.

Mr. Clifford was elected a Junior of the American Society of Civil Engineers on November 8, 1909; an Associate Member on January 15, 1917; and a Member on March 16, 1925.

CHARLES FREDERICK WILSON FELT, M. Am. Soc. C. E.*

Died February 4, 1928.

Charles Frederick Wilson Felt was born on April 29, 1864, at Salem, Mass. He was a descendant of George Felt, who, with John Endicott, came to Salem in September, 1628, and founded, on June 24, 1629, the Town of Charlestown, Mass. (now a part of Boston), named in honor of Charles the First.

The history of New England records the activities of this family and its branches in the settlement and development of the region centering around Casco Bay and Boston. One of Mr. Felt's antecedents was Captain John Felt, who bore an honorable part in the War of Independence against the British, particularly against the latter's aggression at North River Bridge, Salem, wherein they demanded the store of cannon held by the Colonists, on February 26, 1775, culminating in a melee, "which may be justly recorded the first blood of the Revolution."

Charles Wilson Felt, the father, following his graduation from Bridgewater Normal School in 1859, spent many years on the invention of a typesetting machine. Later, he became an author and publisher in Worcester, Mass. He married Martha Seeth Ropes and three children were born to this union.

The early years of Charles Frederick Wilson Felt were spent with relatives at Salem, and at Elizabeth, N. J., because of the death of his mother. Later, he resided at Danvers, Wakefield, Harvard, and Northborough, Mass., with his father, who had re-married. As a young man he worked on farms

* Memoir prepared by a Joint Committee of the Society and the Western Soc. of Engrs. consisting of G. W. Harris and C. A. Morse, Members, Am. Soc. C. E., and J. de N. Macomb, Assoc. M. Am. Soc. C. E.

near Southborough and Bolton, Mass., attending neighboring schools, supplementing family funds with his own efforts, and helping to pay his way through college. He was graduated with the degree of Bachelor of Science from the Massachusetts College of Agriculture at Amherst in 1886.

Work was then his desire and necessity. In September, 1886, he turned to the West where an era of railway construction beckoned. He found his first position as an Axeman on the Atchison, Topeka and Santa Fé Railway, which at that time had an active program of expansion under way in the States of Kansas and Colorado. In this connection, and, subsequently, while with the Denver and Rio Grande, Arizona and Eastern, and the, then, Santa Fé line from Nogales to Guaymas, Mexico, he held the various positions that fell to a young engineer engaged in pioneer railway location and construction.

In May, 1890, he returned to the Santa Fé Company as Resident Engineer at Galveston, Tex., where he remained until July, 1892. Following a brief service on the Rio Grande Southern Railway in Colorado as Office Engineer, he again returned to the Gulf Lines in February, 1893, as Division Engineer, and in May, 1893, he was appointed Resident Engineer of the same line at Galveston. While thus employed, the Texas Railroad Commission, under the so-called Stock and Bond Law, made a valuation of the Gulf Lines of the Santa Fé Railway in which his work, on behalf of the Railway Company, was a conspicuous part.

In September, 1896, Mr. Felt was appointed Chief Engineer of the Gulf, Colorado and Santa Fé Railway Company, with headquarters at Galveston. Shortly thereafter, the Company undertook an extensive program of reconstruction and improvement work, which was planned and executed under his direction. Also, under his supervision, several hundred miles of extensions were constructed in the States of Texas, Oklahoma, and Louisiana. During this time, he also directed the preparation of several important valuations, the need for which arose in connection with the demands of the Railway Commissions of the States of Texas and Oklahoma, as well as the needs of the Company, in connection with rate-case litigation.

In November, 1909, Mr. Felt was appointed Chief Engineer of The Atchison, Topeka and Santa Fé Railway Company, with headquarters at Topeka, Kans. This placed him in charge of directing the engineering features in connection with maintenance and improvement work on the Eastern and Western Lines of the Santa Fé System. The mileage of second track was extended; terminals and shops were enlarged; bridges were strengthened; and the physical plant of the road was generally improved. He also directed and participated in the work of preparing several important rate-case valuations.

In April, 1913, he was appointed Chief Engineer of the entire Santa Fé System, with headquarters at Chicago, Ill. He brought to this last assignment an ability enriched with experience and a high degree of engineering skill. In this position he had charge of all engineering, construction, and valuation work. This embraced the standardization of methods and plans, the use and service of materials and appliances, the development of timber treatment, the location and construction of extensions to existing lines, and the general supervision of engineering matters on the entire system. The

extensions and new lines added to the System amounted to more than 800 miles.

During this time, also, there was initiated and completed the inventory, pricing, conferences, and hearings arising out of the Federal Valuation Act. Mr. Felt's special qualifications and zeal peculiarly fitted him to co-ordinate and direct this work. It is worthy of recital that when he first entered its employ, the Santa Fé operated 5 348 miles of road, which, by the close of his service, had increased to 12 349 miles of main line and branch lines and 1 724 miles of second track, to which increase he had given his best efforts—his all.

Mr. Felt was a Member and a Past Vice-President of the Western Society of Engineers. He was a Charter Member and Past-President of the American Railway Engineering Association, in the work of which he took an active part. He was also a member of the American Railway Association, American Railway Guild, and Past-President of the Chicago Engineers' Club.

He was affiliated with, and was Past-President of the Trustees of, the Hyde Park Presbyterian Church, in Chicago, at the same time retaining his membership in the First Presbyterian Church, which he joined in December, 1893, at Galveston, where the funeral and interment were held.

The following is quoted from a resolution of the Board of Directors of The Atchison, Topeka and Santa Fé Railway Company, passed at its meeting on February 7, 1928:

"*Resolved,* That the Directors express their great sorrow in the death on February 4, 1928, of C. F. W. Felt, System Chief Engineer. * * * Mr. Felt's engineering experience included surveys, location, construction, maintenance, and all engineering problems involved in operation. His ability and his thorough knowledge of engineering, and broad experience, his kindliness to his associates and his fairness in dealing with public authorities, won for him the highest respect and consideration of all. Every one who knew Mr. Felt loved him and feels in his death a personal loss."

He was married on April 6, 1904, to Clara C. Root, of Galveston, who survives him. He is also survived by one brother, Dr. E. Porter Felt, of Nassau, N. Y., two half-brothers, George H. Felt, of Northborough, Mass., and Benjamin F. Felt, of Melrose, Mass., and a half-sister, Elizabeth Ann Felt, of Boston, Mass.

Mr. Felt was elected a Member of the American Society of Civil Engineers on March 3, 1897.

FRANK JAMES HUBBARD, M. Am. Soc. C. E.*

Died February 23, 1927.

Frank James Hubbard was born on February 24, 1868, in Plainfield, N. J. He was the son of Joseph A. and Charlotte Elizabeth (Robertson) Hubbard. He was graduated from the Plainfield High School in the Class of 1887 and

* Memoir prepared by A. W. Vars, M. Am. Soc. C. E.

after his graduation spent nearly three years on a ranch in Wyoming for the benefit of his health.

From 1889 to 1893, Mr. Hubbard was employed in the office of the County Engineer of Union County, New Jersey, on highway construction, surveys for the mapping of Lapeer County, Michigan, and also in charge of street paving in Olean, N. Y. The following year he was a Special Student in the Massachusetts Institute of Technology, at Cambridge, Mass, after which he entered the employ of the City of Plainfield as Assistant Engineer on the construction of its sewer system, serving in that capacity until 1896.

At the conclusion of his engagement with the City of Plainfield, Mr. Hubbard entered private practice, in which he was engaged until the time of his death. During this period he designed a sewer system for the Village of Sea Cliff, Long Island, served as Borough Engineer of North Plainfield, N. J., from 1897 to 1912 and from 1920 to 1926, and as County Engineer of Union County, from 1897 to 1911, designing and supervising the construction of bridges and paving. In 1903, he designed and installed a waterworks system for Alfred, N. Y., and, from 1915 to 1927, held the office of Borough Engineer of Dunellen, N. J. Just prior to his death he completed the work of designing a viaduct over a stream dividing the municipalities of Plainfield and North Plainfield.

Mr. Hubbard was a member of the Union County Chapter of the New Jersey Society of Professional Engineers, the Engineers' Club of Plainfield, and the Rotary Club of Plainfield, of which he was a Charter Member.

He served the community in which he lived in many positions of responsibility and trust and will ever be remembered for his interest and services in civic affairs. He was a member of the Board of Education from 1915 and served as its President from 1923 until his death. During this period, he superintended the erection of a number of the finest school buildings in the city for which work he was eminently fitted by his engineering training and experience. In recognition of his work as a member of this Board, the City honored his memory by giving his name to its most recently constructed school building.

During his life Mr. Hubbard gave much time and attention to Muhlenberg Hospital; he was a member of its Board of Governors for nearly a quarter of a century and its President for four years. He also took an active interest in the building of the newer and larger fireproof structures of this institution, which construction he personally superintended.

He was a member of the Seventh Day Baptist Church and was actively connected with several of its denominational interests. He was Treasurer of the Board of Trustees of the Seventh Day Baptist Memorial Fund which administers the endowment funds of that denomination.

In 1897, Mr. Hubbard was married to Bessie Evelyn Titsworth who survives him.

Mr. Hubbard was elected a Member of the American Society of Civil Engineers on April 12, 1926.

GEORGE NAUMAN, M. Am. Soc. C. E.*

Died September 28, 1927.

George Nauman was born at Lancaster, Pa., on June 20, 1870. He was graduated from Lehigh University in 1890 with the degree of Civil Engineer.

In September, 1890, he entered the service of the Pennsylvania Railroad Company as a Rodman in the Engineering Department. He was successively promoted to the grades of Transitman in September, 1890, Assistant Engineer in February, 1899, and Assistant to the Chief Engineer in July, 1918, which position he held at the time of his death.

In his various capacities, Mr. Nauman had a wide experience in railroad location and construction. It was in the latter work that he found the most enjoyment during his very useful life. An executive, a worker, a co-ordinator, and a doer, he was most content when handling and expeditiously completing the many large construction projects that were under his supervision.

Among the larger projects constructed under his direction were the Delaware River Bridge from Philadelphia, Pa., to Delair, N. J.; the famous Rockville Bridge across the Susquehanna River, at Harrisburg, Pa.; the Shocks Mills Bridge across the Susquehanna River at Shocks Mills, Pa.; the four-tracking of the Northern Central Railroad from York Haven to Marysville, Pa.; the Enola Yards, near Marysville, Pa.; the Northumberland Yard, Northumberland, Pa.; the Montgomery Bridge at Montgomery, Pa.; the Allegheny Division improvements, including the Rockland and Kennerdell Tunnels; the Buffalo Division improvements between Oil City and Corry, Pa.; the Gardenville engine facilities, near Buffalo, N. Y.; the additional engine facilities at Pitcairn and Shire Oaks, Pa.; the Northside improvements, Pittsburgh, Pa., including the Northside track elevation; four-tracking, Pittsburgh to Sharpsburg, Pa.; and the new Sharpsburg Yard and Engine Terminal; the Beck's Run Viaduct; and the Streets Run Viaduct on the Monongahela Division, at Pittsburgh. In addition to these larger projects he planned and constructed many minor railroad improvements, including grade-crossing eliminations and branch-line extensions.

Mr. Nauman's executive superiors held him high in their esteem, recognizing his excellent judgment and abilities. He enjoyed the social side of life during his active career, making many friends who always appreciated his ready wit and repartee. His wise counsel and sympathetic nature won for him the affection of his subordinates who appreciated their contacts with such a fine outstanding character. The profession has sustained a real loss in the passing of Mr. Nauman, a foremost Construction Engineer.

He was a member of the Engineers Society of Pennsylvania, the Harrisburg Club, of Harrisburg, and the University Clubs of Philadelphia, Pa., and of Pittsburgh.

Mr. Nauman was elected a Member of the American Society of Civil Engineers on May 4, 1904.

* Memoir prepared by T. P. Watson, M. Am. Soc. C. E.

STURGIS HOOPER THORNDIKE, M. Am. Soc. C. E.*

Died February 16, 1928.

Sturgis Hooper Thorndike was born on June 11, 1868, at Beverly, Mass., the son of Samuel Lothrop and Anna Lamb (Wells) Thorndike. Of English ancestry, his earliest known paternal ancestor was William Thorndike, who died at Little Carlton, Lincolnshire, England, in 1539. His earliest paternal ancestor in America was John Thorndike, who came from Little Carlton to Ipswich, Mass., in 1633, but who returned to England, leaving in America his son, Paul, who settled in Beverly, Mass. His earliest known maternal ancestor was Harold de Vallibur (*i. e.*, Welles), who lived in Cumberland County, England, early in the Twelfth Century; his descendant, Hugh Wells, came from Essex County, England, to Watertown, Mass., in 1630, later going to Hartford, Conn., and then to Hadley, Mass.

Sturgis Thorndike's boyhood was spent largely in Boston and in Cambridge, Mass. After preparation at a private school, he entered Harvard College, where he received his Bachelor of Arts degree in 1890. Later, having decided to follow an engineering career, he entered the Massachusetts Institure of Technology, from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1895.

Entering the office of the City Engineer of Boston in 1895, at the beginning of a period of notable activity in city bridge construction, he gained during the next eighteen years a broad experience in the municipal engineering service of that city, primarily in its bridge building undertakings. In 1906 he was appointed to the position of Assistant Engineer in charge of bridge design. In 1911, on the consolidation of the Engineering, Street, and Water Departments into the Department of Public Works, he became the Designing Engineer of the Bridge and Ferry Division of this, the largest of Boston's municipal departments. In this position he was in charge of the design of bridges and of other municipal structures.

Among the more important Boston bridges with which Mr. Thorndike had engineering connection, may be mentioned the Charlestown, Summer Street, Northern Avenue, Chelsea Street, Chelsea North, Meridian Street, Broadway, and Atlantic Avenue (or Cove Street) Bridges, all over tide-water and each having an important draw-span; the Cambridge Bridge (now known as the Longfellow Bridge), across the Charles River between Boston and Cambridge, the most notable of the bridges of Greater Boston; and many important bridges on land, spanning railroad locations.

During this period, under leave of absence from the City Engineer during term time, Mr. Thorndike served as Instructor at the Massachusetts Institute of Technology in the scholastic years of 1904-05 and 1905-06, teaching hydraulics, stereotomy, bridge design, and surveying.

* Memoir prepared by Frederic H. Fay and Charles M. Spofford, Members, Am. Soc. C. E.

In October, 1911, he resigned from the position of Designing Engineer of the Bridge and Ferry Division of the City of Boston, to engage for a time in private engineering practice.

On July 1, 1914, Mr. Thorndike joined with Frederic H. Fay, M. Am. Soc. C. E., former head of the Boston Bridge and Ferry Division of the City of Boston, and with Charles M. Spofford, M. Am. Soc. C. E., the head of the Civil Engineering Department of the Massachusetts Institute of Technology, in forming the engineering firm of Fay, Stofford and Thorndike. In this organization he was an active partner until his death.

Among the more important engineering projects under the direction of the firm in which Mr. Thorndike participated, may be mentioned the design and the engineering supervision of construction of the Boston Army Supply Base, one of the larger water-front terminals built during the World War for the use of the United States War Department; the Hampden County Memorial Bridge across the Connecticut River at Springfield, Mass., a monumental structure of reinforced concrete arches; and the water supply, sewage disposal, central heating plant, and other engineering features for the Town of Mariemont, recently built in the suburbs of Cincinnati, Ohio. In all these projects and in many investigations and reports, both engineering and economic, Mr. Thorndike's contributions were valued by reason of his sound judgment and his highly analytical mind.

Thoroughness and painstaking analysis were characteristics of Mr. Thorndike's professional work and, indeed, of whatever he undertook, whether within or without the field of engineering. Inheriting, perhaps, a "legal" mind from his father, who was a lawyer of distinction, he early gave particular attention to the legal form of engineering contracts, and although without special legal training, he became an authority in this important branch of engineering work. Always keenly interested in fair play and the advancement of ethical standards, he was a most active member of the Committee of the Northeastern Section of the Society which formulated the Code of Practice, adopted in 1927 by the Society.

He was long active in the work of the Northeastern Section of the Society, of which he was President at the time of his death. For thirty-two years he had been a member of the Boston Society of Civil Engineers, serving on many of its committees and also as a Director in 1919 and 1920. He was a member of the American Water Works Association, the New England Water Works Association, and the American Society for Municipal Improvements. He frequently contributed papers to these professional societies. He was an active member of the Boston Building Congress, the aim of which is to promote improved conditions in the building industry in Boston and in New England.

Mr. Thorndike was a member of the Harvard Club of Boston, the Harvard Club of New York, and the Harvard Engineering Society. He was long a member of the St. Botolph and Engineers Clubs of Boston and of the Boston City Club, as well as of the Boston Chamber of Commerce.

The Protestant Episcopal Church and its affiliated interests, a vital part of his life, equally with his professional work, claimed much of his time, effort, and energy. From his boyhood—for fifty-two years—when in town, he occupied his place in the family pew at the historic Christ Church in Cambridge. For thirty-five years he was elected to the Vestry, and at one time was a Warden of that parish, of which he was always a most loyal supporter. He was also connected with St. Anne's Church in Lincoln, Mass., near his country home, serving on the Vestry there and giving to that parish his fostering care and intense interest.

Shortly before the war he assisted in organizing the Conference for Church Work, an annual gathering of workers in the Episcopal Church throughout the country and from abroad. Starting in a small way, the organization has outgrown the accommodations of the Harvard Divinity School, where its summer conventions were held for a time, and, in recent years, has met in the early summer at Wellesley College. For many years, Mr. Thorndike was the managing executive of the Conference; through his efforts it was incorporated; and one of the last things he did was to finish the preparation of a code of practice for the conduct of its business.

His interests extended to many enterprises for the betterment of humanity; he was a member of the Fellowship of Reconciliation and of the Church League of Industrial Democracy; he was a member of the New China Committee, and the Japan Committee; he was long active in the support of missionary work in the Protestant Episcopal Church; and he was a liberal contributor, not only to church activities, but also to many other good causes.

For many years Mr. Thorndike made his summer home at Stony Farm, in Weston (and Lincoln), Mass., and during the latter part of his life, Weston was his legal residence. He was interested in the affairs of the town, but never held office until 1926, when he was elected a member of the Water Supply Investigating Committee. In 1927 he was elected a member of the Board of Water Commissioners.

One of his chief characteristics was his love of Nature; he was never more happy than when he was at Stony Farm. He knew and loved every tree on the place and every living creature was his companion and friend. Mt. Monadnock in Southwestern New Hampshire frequently lured him to its slopes, and for many years he was a constant visitor at The Ark, at Jaffrey, at the base of this mountain. One of his pastimes, in which he was often joined by his partner, Mr. Fay, was the surveying and the mapping of the many trails on the slopes of this beautiful isolated mountain.

Mr. Thorndike remained a bachelor throughout his life. A loyal friend and delightful comrade, with qualities of sincerity, unselfishness, and earnestness rarely found, Sturgis Thorndike in his sixty years' span of life exerted a helpful influence among all with whom he came in contact, an influence far greater than he himself ever realized.

Mr. Thorndike was elected a Member of the American Society of Civil Engineers on January 6, 1915.

HOWARD PAUL BAYLY, Assoc. M. Am. Soc. C. E.*

Died August 27, 1927.

Howard Paul Bayly was born October 22, 1884, in Washington, D. C. He was educated in the public schools in the District of Columbia, and was graduated from McKinley Manual Training School in 1903. He received the degree of Bachelor of Science in Civil Engineering from the George Washington University in 1912. In connection with his school and college activities, it should be mentioned that Mr. Bayly took a prominent place in the field of athletics in support of his class interests. He was President of the Engineering Group in his graduating class, and was a member of the Sigma Phi Epsilon Fraternity and President of the Local Chapter. After graduation he followed the development of these same characteristics and held a prominent place in both business and social activities.

Mr. Bayly had varied experience in engineering work of a laboratory nature with Mr. Emile Berliner, in connection with the perfecting of phonograph records, in the United States and Canada. After his graduation from George Washington University he was connected with the Southern Railway Company in construction and engineering work, in charge of its transfer yards at Spencer, N. C., and other important projects in the central southern section of the country. For a time he was Assistant Engineer, with headquarters in Richmond, Va.

In 1918 he left the employ of the Southern Railway Company and joined a contracting firm, Saville and Claiborne, of Richmond. In connection with this firm's work, he was in charge of the residential development which it was carrying on, and he was also engaged in water-works and sewage disposal projects for a number of small cities in Virginia.

With the entry of the United States into the World War, Mr. Bayly was placed in charge of a part of the construction of the cantonment at Camp Lee. Following the completion of this very important project, he was in charge of the DuPont de Nemours housing and industrial project at Hopewell, Va. After the closing of the war projects, Mr. Bayly returned to the building industry in Richmond in association with the firm of Claiborne and Taylor, Contractors, enlarging his activities to include all branches of construction work.

In 1923, he left the firm and engaged in business for himself, confining his activities principally to the very best grade of residences and to more important individual projects rather than to general development work.

Mr. Bayly was married on May 7, 1914, to Alice Burritt, of Washington, D. C., who survives him.

He was a member of Dove Lodge No. 10, A. F. and A. M. The news of his sudden death on August 22, 1927, was keenly felt by his many friends and associates.

Mr. Bayly was elected an Associate Member of the American Society of Civil Engineers on April 16, 1918.

* Memoir prepared by William S. Gordon Dulin, Esq., Charleston, W. Va.

ALFRED LEVY, Assoc. M. Am. Soc. C. E.*

Died March 23, 1928.

Alfred Levy was born at Darlington, England, on April 26, 1878. He received his education in England, after which he served an apprenticeship from September, 1892, to April, 1899, with F. W. Lyall, Civil Engineer, of Darlington, in the surveying and construction of railways, water-works, roads, etc. His services were retained by that firm until April, 1900, when he was engaged as Assistant Engineer under Sir James Bell on the Lancashire, Derbyshire, and East Coast Railway, on maintenance and new works until April, 1903. In May of the same year he was engaged by the Consulting Engineers for the Cape Government Railways (Messrs. Gregory, Eyles and Warring) and sent to South Africa under contract as Assistant Engineer on the survey and construction of the Indwe-Maclear Line (105 miles in length); he was in charge of 30 miles of this road from earthwork to plate-laying. In May, 1906, he went to England on furlough after which he again returned to South Africa.

From February to August, 1907, Mr. Levy was Assistant Engineer on the Cape Government Railways under Mr. M. D. Robinson, and was employed on the Coastal Section of the Mossel Bay-George Line ($33\frac{1}{2}$ miles in length). He had charge of all outside work necessary to complete construction, including setting out, testing of foundations, and the erection of a steel-pile bridge, 650 ft. long. He subsequently returned to England where he was again employed by Mr. E. W. Lyall, of Darlington. Among other engagements he carried out a survey for reclamation work in the Tees Estuary for the Cargo Fleet Ironworks.

In July, 1908, Mr. Levy was appointed Chief Engineer of the Ferrocarril Central Dominicano, Puerto Plata, Dominican Republic (under Mr. H. Gibson). He acted for six months as General Manager, returning to England in 1912.

In January, 1913, he was appointed to the Staff of A. M. Tippett, Engineer-in-Chief of the South African Railways. He was employed for four months on grade and curve improvements on the South-Eastern Line and was then transferred to the Benoni-Welgedacht Line.

When the World War broke out in 1914, Mr. Levy was employed on the survey and construction of the strategical line from Prieska to Upington, in connection with the German Southwest Campaign. During 1915 he was engaged for eight months on reparation work, after serious washouts, on the Bandolier Kop-Messina Line.

In 1916 he was transferred to Ladysmith, Natal, to take charge of the remodeling of the Passenger, Freight, and Locomotive Yard. Heavy earthwork was involved as well as a double-track steel bridge over the Klip River. In 1918 he went to Johannesburg as Assistant Superintendent of Maintenance. The following year he was transferred to Bloemfontein and East London and

* Memoir prepared by Jervis Gibbon, Esq., Dist. Engr., South African Rys. and Harbors, Johannesburg, South Africa.

again to Johannesburg in 1922 in the same capacity. At the time of his death, Mr. Levy had been appointed System Engineer, with headquarters at East London, but had not actually taken up the work.

He always insisted on a high standard of track maintenance and was responsible for numerous improvements for the purpose of eliminating sharp curves and recentering, to allow for transition curves on the sections on which the fast trains ran. His reports on engineering work were concise and his recommendations particularly sound.

He had made a special study of the problem of soil erosion and was most successful in the methods he used for its prevention. His estimates for new works and programs for re-laying, etc., of permanent way were most carefully prepared as to quantities and prices, and local conditions were always allowed for in the price of labor. He had the faculty of preparing plans in such a way as to convey the information required without elaborate or superfluous detail.

In making Staff changes, Mr. Levy was most considerate as to educational facilities for the children of the men concerned. Fortunately for the Engineering Profession, he has handed on these splendid traditions to the younger engineers who have had the privilege of working with him.

On December 31, 1912, Mr. Levy was married to Florence Thomas, of Northallerton, Yorkshire, England, who survives him, together with two children, a daughter aged 14 and a son aged 11. He was essentially a family man, and was devoted to his wife and children.

"So many worlds, so much to do,
So little done, such things to be,
How know I what had need of thee,
For thou wert strong as thou wert true!"

Mr. Levy was elected an Associate Member of the American Society of Civil Engineers on December 5, 1911. He was also a Member of the South African Society of Civil Engineers.